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Support and sealing at the toe of a retaining wall extending to bedrock: theory and practice

Georakentamisen pääaineen diplomityö, joka on jätetty opinnäytteenä tarkastettavaksi diplomi-insinöörin tutkintoa varten.

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Valvoja: Professori Leena Korkiala-Tanttu
Ohjaaja: DI Jaakko Heikkilä

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Tukiseinän alapään vaakastabiliteetin varmistaminen täytyy huomioida kallioon ulottuvissa, tuetuissa kaivannoissa. Kaivuvaiheen aikana vaakastabiliteetti varmistetaan yleisesti tukiseinän alapäähän poratuilla juuritapeilla. Kalliopultit ja juuripalkki asennetaan antamaan lopullista tukea, kun kaivanto on saavuttanut kalliopinnan.
Yleisin Suomessa esiintyvä ongelma juuritappien asentamisessa on tapin väärä kohdistaminen kallioon. Pontinalapään suuri etäisyys kalliotapeista sallii liian suuria seinän alapään siirtymiä.
Toinen käytännön ongelma, joka esiintyy syvissä kaivannoissa, joissa kaivupohjan taso on pohjaveden alla, on vesitiiveyden varmistaminen tukiseinän ja kallion saumakohdissa. Tämä on erityisesti ongelma oloissa, jossa pohjavesivuoto on runsasta kalliopinnan tasolla. Ratkaisuja tarvitaan pohjavesivuotojen vähentämiseksi tukiseinän alapään alueella.
Tämän diplomityön tavoitteena on havaita käytännön ongelmia, jotka liittyvät tukiseinän alapään vaakastabiliteettiin ja tiivistykseen tutkimalla nykyistä käytäntöä. Nykyisen käytännön kartoitus tehtiin haastattelemalla kokeneita insinöörejä ja urakoitsijoita. Haastattelut toteutettiin marraskuusta 2010 toukokuuhun 2011, ja niiden tarkoitus on auttaa ymmärtämään käsitteitä, asenteita ja menetelmiä käytännön prosesseissa.
On huomattava, että tämä diplomityö on tehty Suomessa, ja siten keskittyy Suomen tilanteeseen. Teoria tässä työssä perustuu pääosin Eurooppalaiseen kirjallisuuteen, mutta huomio kiinnitetään niihin osiin, jotka sopivat erityisesti käytettäväksi Suomessa. Haastateltavat koostuivat suomalaisista ammattilaisia. On myös huomattava, että haastattelujen tulokset perustuvat yksinomaan haastateltavien mielipiteisiin ja kokemuksiin.
Tappien kohdistamisen ongelmat liittyvät käytettyyn asennustapaan. Tappien väärä kohdistaminen liittyy porattujen suojaputkien käyttöön. Kalliotappien asentaminen seinään kiinnitetyn putken kautta poistaa tämän ongelman. Seinään kiinnitettyä putkea ei aina suosita käytännössä, johtuen vaikeuksista varmistaa, että putki ei ole vioittunut ponttien asentamisen aikana. Seinään kiinnitetyt putket ovat kalliimpia.
Tiiveyden ratkaisu riippuu työmaan olosuhteista. Juuripalkki on sopiva, kun pohjavesivuoto on vähäistä tai vähäinen vuoto tukiseinien alapään alla on sallittu. Jos pohjavesivuoto on runsasta, tai olosuhteet ovat erittäin herkkiä pohjaveden alenemiselle, on käytettävä injektointimenetelmiä. Suihkuinjektointia pidetään kaikkein toimintavarmimpana menetelmänä. Siksi se on erittäin suositeltavaa niissä tapauksissa, joissa vesitiiveys on kriittistä.

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Avainsanat: ponttiseinän alapää, alapään vaakastabiliteetti, juuritappi, kalliopultti, juuripalkki, injektointi

Aalto University

AALTO UNIVERSITY SCHOOL OF ENGINEERING PO Box 12100, FI-00076 AALTO http://www.aalto.fi	ABSTRACT OF THE MASTER'S THESIS	
Author: Niamh Collins		
Title: Support and sealing at the toe of a retaining wall extending to bedrock: theory and practice		
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The issue of providing lateral support to the toe of a retaining wall arises in supported excavations extending to the bedrock. During the excavation phase lateral support is most commonly provided by rock dowels drilled into the rock along the toe of the wall. Inclined toe bolts and a toe beam are later installed to provide final support at the toe when the excavation has reached bedrock.
Problems relating to the use of rock dowels have been encountered in Finland, in particular problems relating to the incorrect positioning of the rock dowels. A large gap between the wall and the rock dowel allow excessive displacement to occur at the toe.
Another practical problem frequently encountered in deep excavations extending to bedrock where the base of the excavation lies below the groundwater level is the provision of a water tight seal along the base of the wall. This particularly becomes an issue in conditions of high groundwater flow at the level of the bedrock. Solutions must be found to adequately control the flow of groundwater in the region of the toe of the wall.
This thesis aims to address the practical problems relating to the provision of lateral toe support and sealing at the toe through an examination of current practice. This was done by carrying out a series of interviews with experienced engineers and contractors. The interviews were carried out from November 2010 to May 2011 and sought to gain an understanding of the concepts, attitudes and processes used in practice.
It should be noted that this thesis is based in Finland and hence focuses on the situation in Finland. The theory presented in this thesis is mainly based on European literature but attention is drawn to those parts which are particularly relevant for use in Finland. The interviewees consisted of Finnish professionals. It should also be noted that results of the interviews are based solely on the opinions and experiences of the interviewees.
Problems involving the positioning of the rock dowel relate to the method of installation used. Inaccurate positioning of the dowel is associated with the use of bored installation tubes. Installation of the rock dowels through casings attached to the wall eliminates this problem. However pre-attached casings are not always favored in practice due to the difficulties in ensuring the casing is not damaged during driving. Additionally pre-attached casings are a more expensive method.
The solution for sealing at the toe depends on the site conditions. A toe beam is suitable for conditions of low groundwater flow where the occurrence of some seepage beneath the toe is allowable. For conditions of higher groundwater flow or conditions very sensitive to lowering of the groundwater grouting techniques must be employed. Jet grouting is considered the most dependable technique and therefore is highly recommended where sealing at the toe is critical.

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Preface

Two important considerations particular to the case of a retaining wall extending to bedrock include the provision of lateral support at the toe of the retaining wall and control of seepage beneath the toe. This thesis seeks to examine both of these aspects, considering the theoretical background and looking particularly at issues and solutions found in practice.

I would like to thank all those involved in this thesis: firstly of all to those who agreed to be interviewed and gave their time to share their experiences; to Vesa Oksanen and Iikka Kärki for helping with interview process, providing translation and guidance; to all my colleagues at Finnmap Infra Oy for their help and support; to Leena Korkiala-Tanttu, Pauli Vepsäläinen and Jaakko Heikkilä for their advice and for overseeing the project; and finally to Länsimetro Oy for providing funding for this thesis.

Espoo 21.11.2011

Niamh Collins

Table of Contents

List of Symbols

A_N	[m^2]	area of the rock dowel cross-section carrying axial loads
A_{dowel}	[m^2]	cross-sectional area of rock dowel
D	[m]	estimated distance between the toe of the wall and the rock
F_f	[kN/m]	friction force along the toe
F_v	[kN/m]	vertical axial force at the toe
M_{Ed}	[kNm]	maximum bending moment on the rock dowel
$M_{pl,Rd}$	[kNm]	plastic moment capacity of the rock dowel
$M_{pl,Rd,N}$	[kNm]	plastic moment capacity reduced for axial loads
N_{pl}	[kN]	axial capacity of the rock dowel
V_{Ed}	[kN]	design horizontal load or design shear force per rock dowel
$V_{pl,\ Rd}$	[kN]	horizontal load or plastic shear force capacity of rock dowel
V_{Rd}	[kN]	horizontal load or shear force capacity of rock dowel
W_{el}	[m^3]	elastic section modulus
W_{pl}	[m^3]	plastic section modulus
$W_{pl,N}$	[m^3]	plastic section modulus reduced for axial loads
c	[m]	spacing of the rock dowels
d	[m]	rock dowel diameter
f_{ck}	[kN/m^2]	concrete cubic strength
f_y	[kN/m^2]	yield strength of steel
f_{yd}	[kN/m^2]	design yield strength of the steel
q_{Ed}	[kN/m]	design horizontal load along the toe of the wall
γ_n	[-]	partial safety factor depending on the safety class
γ_m	[-]	material partial safety factor
Δ	[m]	effective gap
η	[-]	shape factor
μ	[-]	coefficient of friction between the toe and the rock surface
σ_{max}	[kN/m^2]	maximum allowable tensile stress
σ_{ult}	[kN/m^2]	ultimate strength of the steel
τ_{max}	[kN/m^2]	maximum allowable shear stress

1 Introduction

Deep excavations in soft soil extending to bedrock are commonly supported using an anchored sheet pile supported laterally at the toe by rock dowels drilled into the bedrock. Rock dowels are installed prior to excavation and so provide immediate support during excavation and on reaching the final dig level. Problems relating to the use of rock dowels have been encountered in Finland. In particular the incorrect installation of the rock dowels has led to serious deficiencies of the lateral support at the toe of the wall. In these cases the dowels were positioned quite far from the wall, such that the resulting large gap between the wall and the rock dowel allowed excessive displacement to occur at the toe.

Another practical problem frequently encountered in deep excavations extending below the groundwater level is the control of groundwater flow into the excavation. In excavations extending to bedrock the provision of a watertight seal along the base of the wall must also be considered. This particularly becomes an issue in conditions of high groundwater flow at the level of the bedrock. Solutions must be found to adequately control the flow of groundwater in the region of the toe of the wall.

This thesis aims to address the practical problems relating to the provision of lateral toe support and sealing at the toe. These issues were considered in relation to the theoretical background described in literature as presented in Chapter 2 and Chapter 4. The current practice in Finland was examined by carrying out a series of interviews with experienced engineers and contractors. The interviews were carried out from November 2010 to May 2011 and sought to gain an understanding of the design concepts and construction processes used in practice and the attitudes towards these concepts and processes. An overview of the outcome of these interviews is presented in Chapter 3 and Chapter 5. Finally a case study of Urheilupuisto metro station is presented in Chapter 6. This case study considers the practical application of the matters discussed in the preceding chapters.

It should be noted that this thesis is based in Finland and hence focuses on aspects most relevant to excavations in Finland. The theory presented in Chapter 2 and Chapter 4 is mainly based on European literature but attention is drawn to those parts which are particularly applicable for use in Finland. The interviews were conducted with Finnish engineers and contractors. Therefore the outcome of these interviews given in Chapter 3 and Chapter 5 reflects the current situation in Finland.

It should also be noted that results of the interviews are based solely on the personal opinions and experiences of the interviewees. In order to allow the interviewees to speak freely the information given in Chapter 3 and Chapter 5 is not credited to particular people. It was found that there was broad agreement between the interviewees on most topics. The few differing opinions encountered during this series of interviews are highlighted in the relevant chapters.

2 Lateral toe support: Theory

2.1 Overview

The issue of providing lateral support to the toe of a retaining wall arises in supported excavations extending to the bedrock or to such a level that the depth from the dig level to the bedrock is quite shallow. In such cases horizontal restraint is not provided by passive earth pressure at the toe. Other means of support must be implemented which are capable of transferring the horizontal loads at the toe of the retaining wall to the bedrock.

During the excavation phase lateral support is most commonly provided by rock dowels drilled into the rock along the toe of the wall as illustrated in Figure 1. These will be dealt with in detail in this chapter and the next. The inclined toe bolts and toe beam, also shown in Figure 1, are used to provide support in the final state when the excavation has reached bedrock. (Suomen Rakennusinsinöörien Liitto. 1989). These and other relevant design considerations will also be discussed.

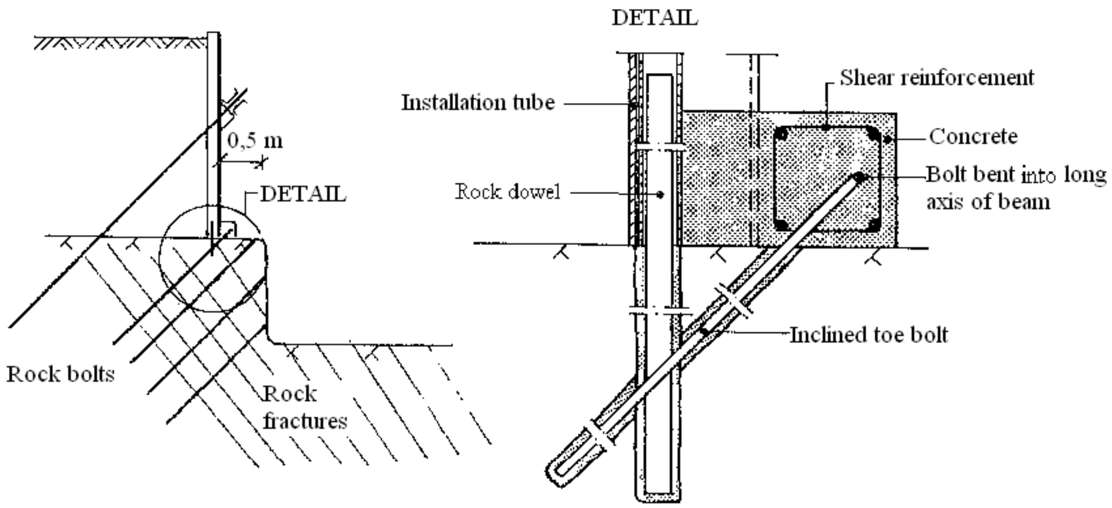


Figure 1. Detail of rock dowels, inclined toe bolt and toe beam. (Suomen Rakennusinsinöörien Liitto. 1989)

2.2 Driving of sheet piling

As will be seen it is important for the capacity of the toe support that the retaining wall is driven as completely as possible down to the bedrock. This minimizes the gap between the toe of the wall and the bedrock (Suomen Rakennusinsinöörien Liitto. 1986). It also ensures that any axial forces carried by the wall are transferred directly to the rock (Jensen, 2008). Both of these factors are important for maximizing the capacity of the rock dowels. The depth of penetration required to reach bedrock during pile driving can be estimated from a rock surface model drawn up during the site investigation. Failure to drive each profile to the rock is more problematic for walls of a short length which have less redundancy than longer walls.

However even if a sheet pile profile is driven fully to rock it is likely that contact between the sheet pile and the rock will exist only at that point along the flat lower edge of the sheet pile which first encounters the rock. This is especially pronounced at a steeply sloping rock surface. In order to increase the length of contact the lower edge of the sheet pile may be chamfered to conform to the contours of the rock surface as shown in Figure 2 (Jensen, 2008). An estimation of the chamfering required is based on an accurate rock surface model which closely maps the slopes of the bedrock surface.

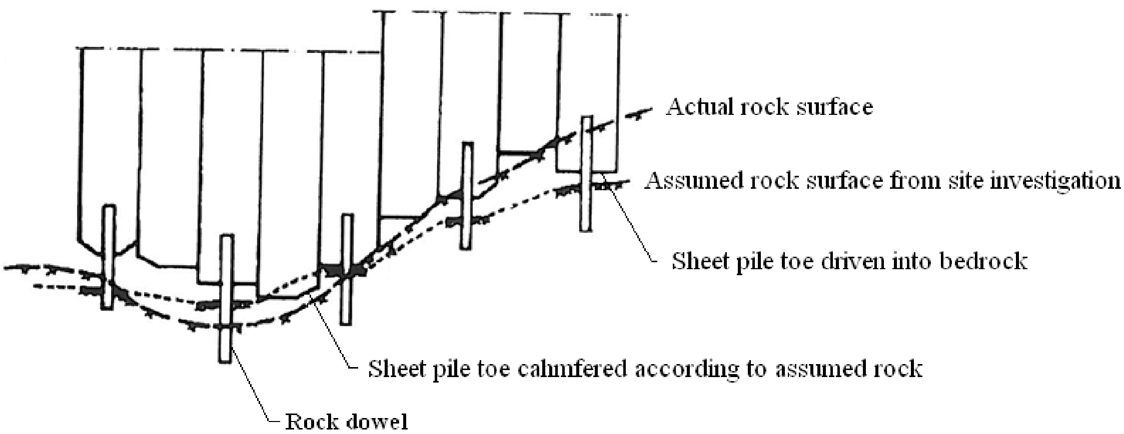


Figure 2. Chamfering of the sheet pile toe and driving of the sheet pile toe into bedrock. (Jensen, 2008)

In certain conditions, such as heavily broken rock surfaces or soft rock types, it is possible to drive the sheet pile into the rock to some extent as shown in Figure 2. This technique gives the advantages of ensuring good rock contact and providing extra capacity for the transfer of loads directly from the retaining wall to the rock. However this technique can be rather expensive and time consuming. Moreover it increases the risk of damage to the sheet pile toe, to the interlocks, and to any steel tube casing attached to the wall for the installation of rock dowels, due to the very high forces used in driving against rock. Local yielding of the steel at the toe and a decrease of shear transfer capacity between piles due to declutching of the interlocks can lead to progressive failure of the wall system. Crushing or bending of the casing causes difficulties for the drilling and installation of rock dowels through the casing. (Jensen, 2008)

For conditions where such excessive damage is likely, as in regions of hard intact bedrock, an alternative might be to blast a channel into the bedrock, providing a seat into which the toe of the wall can be driven. Small quantities of explosives are lowered down into the rock through boreholes drilled from the ground surface along the proposed line of the sheet pile wall. The detonation of these explosives fragments the rock such that the sheet pile may be driven into this highly fractured zone. The channel may alternatively be formed by drilling a row of overlapping boreholes through the soil layers and into the rock. (Technical European Sheet Piling Association. 2001).

2.3 Rock dowels

2.3.1 Installation and general arrangement

As previously mentioned the use of rock dowels is the preferred method for providing lateral support at the toe. Rock dowels are installed prior to the removal of the soil from the excavation and so offer immediate support during excavation and on reaching the final dig level. In this they contribute to safer building practices on site. In addition their confined location at the base of the wall does not limit the working space inside the excavation (Rakennustietosäätiö RTS. 2010).

The rock dowels themselves consist of high grade circular steel bars, 50 - 100 mm in diameter (Suomen Rakennusinsinöörien Liitto. 1986). They are installed by either of two methods: through a casing attached to the wall or through a bored installation tube.

The installation of rock dowels through a casing attached to the wall, as shown in Figure 3, is the most widely recommended method in literature. It is favored as it provides accurate positioning of the rock dowel. It is the only method described in Eurocode SFS EN 12063:1999 Execution of special geotechnical work. Sheet pile walls.

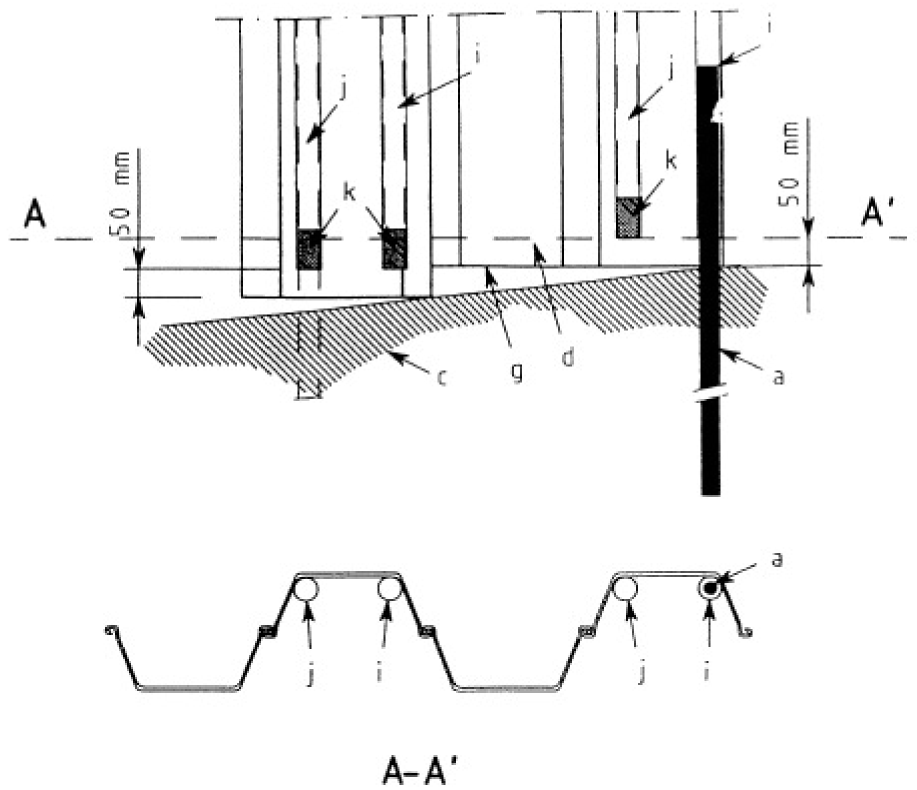


Figure 3. Detail of rock dowels installed through a casing attached to the wall: a = rock dowel, c = bedrock, d = sheet pile, g = level of sheet pile toe, i = casing placed where the distance from the sheet pile to the rock is expected to be the smallest, j = spare casing, k = concrete plug to be put in before installation (SFS EN 12063:1999)

Prior to driving a steel tube casing is welded to the excavation side of the sheet pile profile or to the vertical beam of a soldier and lagging wall (Suomen Rakennusinsinöörien Liitto. 1989). Preferably it should be positioned at the point of the profile which is expected to first encounter the bedrock, especially where the rock surface slopes steeply.

This will minimize the gap between the toe of the wall and the bedrock at the location of the dowel (Kort, Karlsrud, 2008).
The end of the casing should be located at least 50 mm above the toe of the profile in order to avoid damage to the casing while driving the toe against rock (SFS EN 12063:1999). A concrete plug is placed inside the lower 0,5 - 1,0 m length of the casing with the purpose of both preventing soil from entering the casing and avoiding damage to the casing during driving (Liber, Stockholm. 1984). This plug is drilled through when installing the rock dowels. If the soil conditions are particularly difficult for driving, for example in very stiff or stony soils, further protection may be provided by welding a steel angle to the wall so that it covers the casing, as shown in Figure 4 (Jensen, 2008).

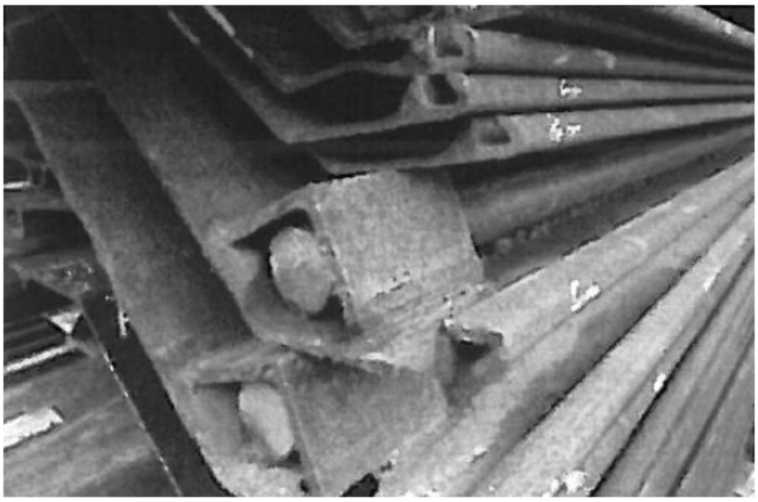


Figure 4. Steel angle covering the end of a casing attached to the sheet pile profile. (Jensen, 2008)

The second method of installation is carried out through an installation tube bored through the soil down to bedrock after the sheet pile wall has been put in place. The tube is to be located in front of and as close to the wall as possible in order to minimize lateral displacement of the toe (Suomen Rakennusinsinöörien Liitto. 1974). However, this method does not necessarily provide the close-fitting restraint offered by the use of a casing attached to the wall (Ryner, Fredriksson, Stille, 1996).

Using either of these methods, a hole is drilled into the rock through the casing or bored installation tube for the placement of the rock dowel. The diameter of this hole should be at least 3 mm larger than the rock dowel diameter. It is then flushed clean and filled with grout through a hose fed down to the bottom of the hole before the rock dowel is installed. Care must be taken to ensure that the dowel has been placed fully down to the bottom of the hole and it may be necessary to push the dowel down fully using the drilling equipment (Ryner, Fredriksson, Stille, 1996). The rock dowels are grouted at least 0,5 m into intact rock (Suomen Rakennusinsinöörien Liitto. 1974), though longer embedment lengths of up to 2 m may be more suitable depending on the quality of the rock. A sufficient quantity of grout should be used so that the dowel is grouted right up to the surface of the rock and a solid plug of grout is formed at the rock surface below the toe. Thus full fixity of the rock dowel in the rock is ensured and the effective gap is minimized. (Jensen, 2008)

Rock dowels extend at least 0,5 m above the rock level (Suomen Rakennusinsinöörien Liitto. 1974). To reduce the horizontal displacements of dowels installed through a casing attached to the wall a small quantity of grout may be injected into the casing filling the space between the dowel and the wall of the tube (Ryner, Fredriksson, Stille, 1996). Grease may be applied to the portion of the dowel inside the casing to prevent bonding or fiction between the dowel and the grout or the concrete plug in the casing. This both prevents vertical loads from the wall being introduced into the dowels and facilitates easier extraction of the sheet pile profiles after their use (Jensen, 2008).

The dowels must be installed at least at every second profile (Liber, Stockholm. 1984). It is common to provide a dowel at each profile. In soil conditions likely to hinder attempts to drive the sheet pile fully to bedrock, where large gaps between the toe of the wall and the bedrock are to be expected, provision for extra support capacity at the toe can be made by allowing for more than one dowel per profile. If the dowels are to be installed through a casing attached to the wall, the extra casings must be welded to the wall prior to driving as indicated in Figure 3 (SFS EN 12063:1999).

For cut-off retaining walls such as slurry walls or secant pile walls the installation tubes are attached to the reinforcement cage. The rock dowels are located and cast within the cross section of the wall as shown in Figure 5. The wall is in bending under the horizontal earth pressures and the restraining forces. Close to the toe this will cause the side of the wall on the excavation side to be in compression while the side against the retained earth will be in tension. The rock dowels for the transfer of the horizontal toe loads by shear are placed on the compression side. Rock dowels are also placed on the side of the wall in tension and are designed to resist pull-out forces only. (Suomen Rakennusinsinöörien Liitto. 1989)

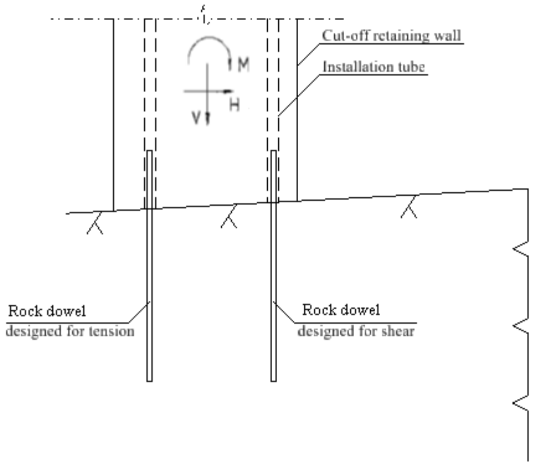


Figure 5. Section through the base of a cut-off wall connected to the rock by rock dowels where H is the horizontal force due to earth pressure behind the wall, V is the vertical force, M is the overturning moment.

2.3.2 Design

The common design approach for the toe support assesses the capacity of the rock dowels only. It is assumed that grouting has been properly carried out to provide a fully fixed connection and that the rock is of sufficient quality to carry the loads from the rock dowels (Suomen Rakennusinsinöörien Liitto. 1989).

The simplest design considers the rock dowels under shear forces only. The shear force per dowel is equal to the horizontal load per dowel and is given by (Ryner, Fredriksson, Stille, 1996):

$$V_{Ed} = q_{Ed} c \qquad (1)$$

where
V_{Ed} is the design horizontal load and the design shear force per rock dowel [kN]
q_{Ed} is the design horizontal load along the toe of the wall [kN/m]
c is the spacing of the rock dowels [m]

A common design used in Finland is given in Pohjarakennus RIL 95 (Suomen Rakennusinsinöörien Liitto. 1974), publish in 1974 and is restated in Rakennuskaivanto-ohje RIL 181-1989 (Suomen Rakennusinsinöörien Liitto. 1989). This design makes use of an equation for the shear capacity of the connection between the elements of a composite beam. The composite beam consisted of a steel section connected to a concrete slab by a steel dowel (Schlaginhaufen, 1965). Thus the shear capacity of the connection at the toe of wall according to Pohjarakennus RIL 95 is given by the minimum of:

$$V_{Rd} = 3 \cdot 10^3 \cdot d^2 \sqrt{10{,}19x10^3 \cdot f_{ck}}$$
$$or \tag{2}$$
$$V_{Rd} = \frac{\pi d^2}{20} \cdot \sigma_{ult}$$

where V_{Rd} is the shear force capacity of the rock dowel [kN]
d is the rock dowel diameter [m]
f_{ck} is the cubic strength of the grout [kN/m^2]
σ_{ult} is the ultimate strength of the steel [kN/m^2]

The first part of equation (2) relates to the capacity of the grout anchoring the rock dowel into the rock. The second part of equation (2) relates to the shear capacity of the rock dowel itself. The capacity of both the grout and the rock dowel are reduced by a factor of safety of 5 and this is included in equation (2). This high factor of safety is required for the difficult working conditions within the excavation. (Suomen Rakennusinsinöörien Liitto. 1974)

The connection is sufficient when:

$$V_{Ed} < V_{Rd} \tag{3}$$

To assume that the dowel is in shear only requires that there is full contact between the toe of the retaining wall and the rock at the location of the dowel and that the dowel is fully fixed up to the surface of the bedrock. This can be assumed to be true for cut-off walls such as slurry walls and secant pile walls. For such walls, it is possible to remove all the soil overlying the bedrock and to cast the concrete of the wall flush with the rock surface. However for driven sheet pile walls it cannot always be assumed that there is full contact between the sheet pile toe and the rock at the location of the rock dowel as soil conditions may prevent driving of the sheet pile fully to bedrock. The large factor of safety included in equation 2 allows a gap of up to 100 – 200 mm between the sheet pile toe and bedrock. If the gap is found to be larger than this then the rock dowels are to be deemed an insufficient solution and temporary reinforcement is required at the toe of the wall before the installation of the final lateral support at the toe. (Suomen Rakennusinsinöörien Liitto. 1974)

An alternative design procedure recommends that the bending moment resulting from the gap between the toe of the sheet pile and the bedrock, shown in Figure 6, is checked directly (Ryner, Fredriksson, Stille, 1996). The slender section of the rock dowels make them particularly weak in bending To determine the bending moment acting on the rock dowel the horizontal toe load, calculated according to equation (1), is applied as a point load acting at the level of the toe of the wall.

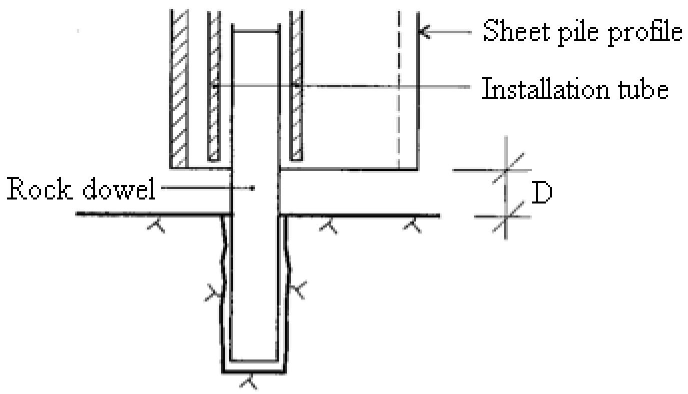


Figure 6. Gap between the toe of the sheet pile and the rock surface. (Ryner, Fredriksson, Stille, 1996).

The moment distribution along the length of the rock dowel depends on the connection of the dowel both to the rock and to the wall. A dowel installed through a casing attached to the wall may be modeled as having full rotational stiffness at both ends and horizontal restraint at the level of the point of fixity in the rock. This is described as the "fixed" model and is shown in Figure 7. The maximum moment occurs at both ends of the dowel and is given by (Dowling, Knowles, Owens, 1998):

$$M_{Ed} = 0{,}5 V_{Ed} \Delta \quad (4)$$

where M_{Ed} is the maximum bending moment on the rock dowel [kNm]
V_{Ed} is the horizontal force per dowel [kN]
Δ is the effective gap [m]

If full rotational stiffness cannot be assumed at one end, then that end is modeled as pinned (Kort, Karlsrud, 2008). This is described as the “free” model and is shown in Figure 7. This model is relevant if the rock or grout is not deemed to be of sufficient strength to provide a fully fixed connection or if the dowel is not anchored properly inside the casing. In the case of a rock dowel installed through a bored installation tube there is no connection to the sheet pile at the upper end of the dowel. Thus it is modeled as a cantilever, relying solely on the anchorage in the rock to provide rotational restraint (Suomen Rakennusinsinöörien Liitto. 1974). This model is also shown in Figure 7. The maximum moment acting on the rock dowel is similar in both of these cases and it occurs at the end providing the rotational restraint as shown in Figure 7. The maximum moment is given by (Dowling, Knowles, Owens, 1998):

$$M_{Ed} = V_{Ed} \Delta \quad (5)$$

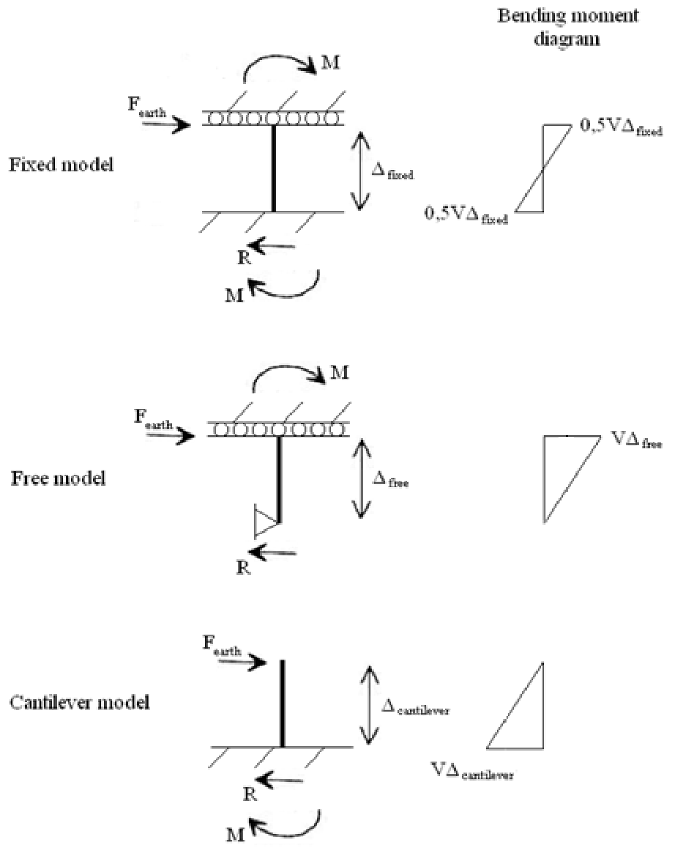


Figure 7. Fixed model, free model and cantilever model of a rock dowel and their associated bending moment diagrams, where M = rotational restraint, R = horizontal restraint, F_{earth} = lateral loads at the toe, V = horizontal force per rock dowel, Δ is the effective gap.

The effective gap assumed for the design should be chosen carefully taking into account both site conditions and the fixity of the rock dowel. Drivability of the sheet pile in the soils on site and the slope of the rock surface influence the size of the gap. The effective gap of a dowel modeled with a fixed end is smaller than that of a free-end modeled dowel (Kort, Karlsrud, 2008).

More generally though, it is advised that a minimum effective gap of 60 mm is used (Liber, Stockholm. 1984). The effective gap may then be taken to be (Ryner, Fredriksson, Stille, 1996):

$$\Delta = D + 60mm \tag{6}$$

where Δ is the effective gap [m]

D is the estimated gap between the toe of the wall and the rock [m]

The bending moment capacity of the rock dowel may be calculated utilizing the plastic bending capacity (Karlsrud, Gjelsvik, Loo, 2004). By plastic analysis yielding is permitted to occur across the entire section as shown in Figure 8.

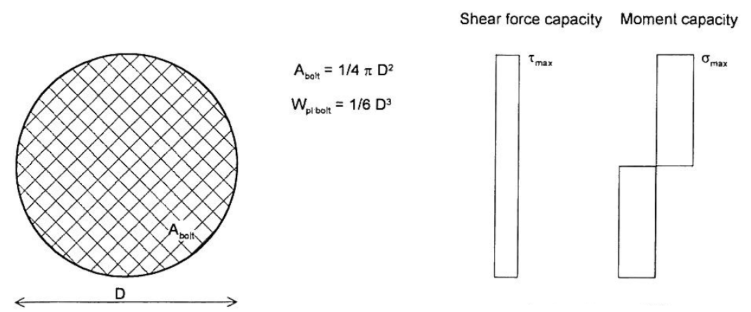


Figure 8. Plastic moment and shear distribution over the cross-section of a rock dowel, where τ_{max} is the maximum allowable shear stress, σ_{max} is the maximum allowable tensile stress. (Jensen, 2008)

Such a plastic moment design is presented in the Swedish Sponthandboken (Ryner, Fredriksson, Stille, 1996). This design applies partial factors of safety to the material properties, in this case to the yield strength of the steel. The moment capacity of the rock dowel is given by (Ryner, Fredriksson, Stille, 1996):

$$M_{pl,Rd} = W_{pl} f_{yd} \tag{7}$$

where $M_{pl,Rd}$ is the plastic bending moment capacity of the rock dowel [kNm]

W_{pl} is the plastic section modulus of the rock dowel [m^3]

$$W_{pl} = \eta W_{el}$$

where η is the shape factor, which is 1,7 for a solid round dowel [-]

W_{el} is the elastic section modulus of the rock dowel [m^3]

f_{yd} is the design yield strength of the steel [kN/m^2]

$$f_{yd} = \frac{f_y}{\gamma_n \gamma_m}$$

where f_y is the yield strength of the steel [kN/m^2]

γ_n is the partial safety factor depending on the safety class and is 1,2 for deep excavations [-]

γ_m is the material partial safety factor and is 1,6 – 2,0 [-]

The connection is sufficient when (Ryner, Fredriksson, Stille, 1996):

$$M_{Ed} < M_{pl,Rd} \tag{8}$$

Rewriting equation (7) in term of horizontal load for each of the cases of moment distribution described above in equations (4) and equation (5), we get

$$V_{pl,Rd} = \frac{2\eta W f_{yd}}{\Delta} \text{ ...fixed model} \tag{9}$$

$$V_{pl,Rd} = \frac{\eta W f_{yd}}{\Delta} \text{ ...free, cantilever model} \tag{10}$$

Thus for a given effective gap distance and a given rock dowel diameter, the "free" model and the cantilever model provide half of the load transfer capacity provided by the "fixed" model.

However equation (7), and hence equations (9) and (10), may lead to an overestimation of the rock dowel capacity as they neglect to take into account the effect of shear on the plastic moment capacity. When the shear force is less than half of the shear capacity of the section, the effect of the shear on the plastic moment capacity is negligible and can be ignored. However when shear is dominant across the section, that is, when the shear force exceeds half of the shear capacity of the section, the plastic moment capacity must be reduced.

The shear capacity is calculated according to plastic distribution of shear stresses, assuming even distribution of shear stresses across the section as shown in Figure 8 and is given by (Ryner, Fredriksson, Stille, 1996):

$$V_{pl,Rd} = \frac{\pi d^2}{4} \cdot \frac{f_y}{\gamma_n \gamma_m \sqrt{3}} \tag{11}$$

where $V_{pl,\,Rd}$ is the shear force capacity of the rock dowel [kN]

d is the rock dowel diameter [m]

f_{yd} is the design yield strength of the steel [kN/m^2]

$$f_{yd} = \frac{f_y}{\gamma_n \gamma_m}$$

where f_y is the yield strength of the steel [kN/m^2]

γ_n is the partial safety factor depending on the safety class and is 1,2 for deep excavations

γ_m is the material partial safety factor and is 1,6 – 2,0

According to the Von Mises yield criterion, the maximum allowable tensile stress and the maximum allowable shear stress across a section are related to the yield strength by:

$$f_y = \sqrt{\sigma_{\max}{}^2 + 3\tau_{\max}{}^2} \tag{12}$$

where f_y is the steel yield strength [kN/m^2]

σ_{max} is the maximum allowable tensile stress [kN/m^2]

τ_{max} is the maximum allowable shear stress [kN/m^2]

Considering a combination of shear and bending across the section, the shear capacity is then given by:

$$V_{pl,Rd} = \frac{\pi d^2}{4} \cdot \tau_{\max} \tag{13}$$

and the bending capacity is given by:

$$M_{pl,Rd} = W_{pl} \sigma_{\max} \tag{14}$$

Then the maximum horizontal load which can be carried by the rock dowels is given by (Kort, Karlsrud, 2008):

$$V_{pl} = \sqrt{\frac{4A_{dowel}{}^2 f_{yd}{}^2 W_{pl}{}^2}{A_{dowel}{}^2 \Delta^2 + 12W_{pl}{}^2}} \text{ …fixed model} \tag{15}$$

$$V_{pl} = \sqrt{\frac{A_{dowel}{}^2 f_{dy}{}^2 W_{pl}{}^2}{A_{dowel}{}^2 \Delta^2 + 3W_{pl}{}^2}} \text{ …free model} \tag{16}$$

where A_{dowel} is the cross-sectional area of the rock dowel (m^2)

$$A_{dowel} = \frac{\pi d^2}{4}$$

It is also important to note the effect of axial loads in reducing the plastic capacity. The cross-section of the bolt is divided into two areas, one designated to carry axial stresses and one designated to carry bending stresses. The shear force is distributed across the entire section as before and the plastic shear capacity is as given in equation (13).

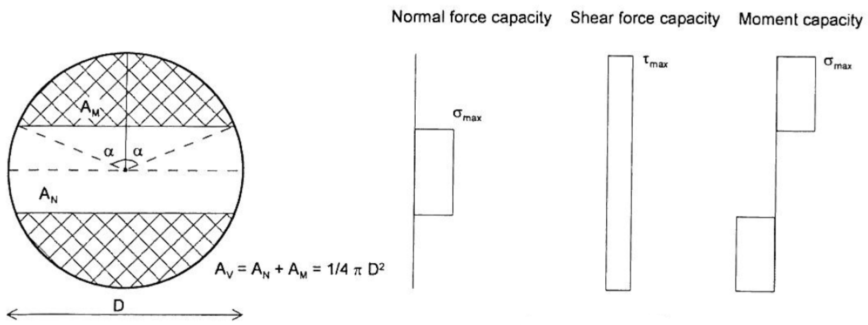


Figure 9. *Plastic moment and shear and axial stress distribution over the cross-section of a rock dowel. (Jensen, 2008)*

The plastic axial capacity is given by:

$$N_{pl} = A_N \sigma_{max} \tag{17}$$

where N_{pl} is the axial capacity of the rock dowel [kN]

A_N is the area of the dowel carrying axial loads [m^2]

σ_{max} is the maximum allowable stress [kN/m^2]

The plastic bending capacity is given by:

$$M_{pl,Rd,N} = W_{pl,N}\sigma_{\max} \tag{18}$$

where $M_{pl,Rd,N}$ is the plastic moment capacity reduced for axial loads [kNm]
$W_{pl,N}$ is the plastic section modulus reduced for axial loads [m^2]
σ_{max} is the maximum allowable tensile stress [kN/m^2]

Using the Von-Mises failure criterion given in equation (12), the maximum horizontal load is given by (Jensen, 2008):

$$V_{pl} = \sqrt{\frac{4A_{dowel}{}^2 W_{pl,N}{}^2 (A_N{}^2 f_y{}^2 - N_{pl}{}^2)}{A_N{}^2 (A_{dowel}{}^2 \Delta^2 + 12W_{pl,N}{}^2)}} \quad \text{…fixed model} \tag{19}$$

$$V_{pl} = 0.5\sqrt{\frac{4A_{dowel}{}^2 W_{pl,N}{}^2 (A_N{}^2 f_y{}^2 - N_{pl}{}^2)}{A_N{}^2 (A_{dowel}{}^2 \Delta^2 + 12W_{pl,N}{}^2)}} \quad \text{…free model} \tag{20}$$

Therefore, in order to maximize the capacity of the connection, it is recommended that axial loads area not to be carried by the rock dowels. Instead axial loads are transmitted by direct contact between the toe of the wall and the rock.

The deformations of the rock dowels are generally not calculated. It has been found from experience that when the dowels are sized according to the equations given above the deformations are within a few millimeters. (Ryner, Fredriksson, Stille, 1996).

2.4 Failure of the rock

The calculations given in the previous section were based on the assumption that the rock is of good quality and has adequate strength to carry the loads from the rock dowels. The grout must also be specified and poured correctly to ensure adequate strength. This is particular important for dowels designed as cantilevers which depend on the connection in the rock for full rotational stiffness. For dowels designed with the assumption that rotational stiffness is provided at both ends, a loss of end fixity due to failure of the rock or grout leads to a 50% reduction in the load capacity of the rock dowel as can be seen from the equations (9) and (10). Additionally the flexibility introduced at the connection causes the effective gap to increase. The capacity of the rock dowel is particularly sensitive to an increase of the effective gap at smaller gaps. A combination of these factors can reduce the capacity of the rock dowels such that the horizontal toe loads exceed the rock dowel capacity (Kort, Karlsrud, 2008). Therefore the ability of the rock to provide restraint for the rock dowels and to carry the loads from the rock dowels must be confirmed for design during the site investigation. If it is found that the rock is highly fractured either naturally or due to previous blasting activity, the quality may be improved with injection grouting (Suomen Rakennusinsinöörien Liitto. 1974).

Figure 10. Highly fractured rock beneath the toe of a sheet pile wall. Photo: Tapio Ranta-Aho

2.5 Failure of the sheet pile

Recent research carried out by the Norwegian Geotechnical Institute has found that a design based solely on the capacity of the dowels may be deficient as it neglects the influence of the sheet piles on the load transfer capacity at the toe (Karlsrud, Gjelsvik, Loo, M. M. 2004). It is important to note that this research was based on the behavior of AZ Arcelor Mittal profiles with Larssen type interlocks and so should not be assumed to be generally valid. However, the findings are still of interest to the topic of this thesis.

The research carried out by the Norwegian Geotechnical Institute found that while the capacity of the rock dowels governs when the gap between the toe of the sheet pile and the bedrock is large, yielding in the sheet pile may control the load transfer capacity of the connection for smaller gaps. The lateral support cannot then be improved simply by increasing the rock dowel diameter or steel yield strength as the equations in the section 2.3.2 would suggest. Instead the dimensions and strength of the piling also need to be taken into account. (Karlsrud, Gjelsvik, Loo, 2004)

Local yielding near the toe mostly occurs at the corners of the sheet pile profile and along the connection between the wall and the rock dowel or the casing. Yielding at the toe causes the effective gap to increase and so it may be beneficial to reinforce the toe with welded steel plates. (Jensen, 2008)

Overall stability of a sheet pile supported by rock dowels may be checked by modeling the toe of the sheet pile as pinned so that bending moments at the toe are zero. This is representative of the case where the toe support is not directly attached to the wall. If the toe support is fixed to the wall, this model ignores the stiffness imparted by the connection between the rock dowel and the wall. The assumed pinned connection is easier to model, however, and results in a more conservative design, making it a suitable design assumption (Karlsrud, Gjelsvik, Loo, 2004).

During the research work carried out by the Norwegian Geotechnical Institute on AZ profiles it was noted that declutching of the interlocks close to the toe support may occur. This resulted in a decrease in the capacity of the connection at the toe. The occurrence of declutching is influenced by the position of the rock dowel in relation to the interlock. The dowel should preferably be placed on the side of the interlock such the resultant force from the rock dowel and the active earth pressure acting on the wall press both sides of the interlock towards each other, giving the most stable configuration. This is illustrated in Figure 11, Case 1. Locating the dowel on the side of the interlock such that these forces are conducive to opening of the interlock, as shown in Figure 11, Case 2 may be required in steeply sloping rock to decrease the effective gap. In this case welding of the interlock nearest the dowel can minimize or prevent declutching. The weld is placed over a length of the interlock at the same elevation as the lower half of the bolt's connection with the wall, usually 0,5m long. The weld is designed to transfer the shear loads acting along the edge of the profile across the interlock. (Kort, Karlsrud, 2008)

It is only possible to place this weld prior to driving so that both profiles are subsequently driven together. If this is not possible, an estimate of the shear transfer capacity of the interlock is made, generally recommended to be no more than 500 kN and that declutching of a few centimeters is allowed in the design. (Jensen, 2008)

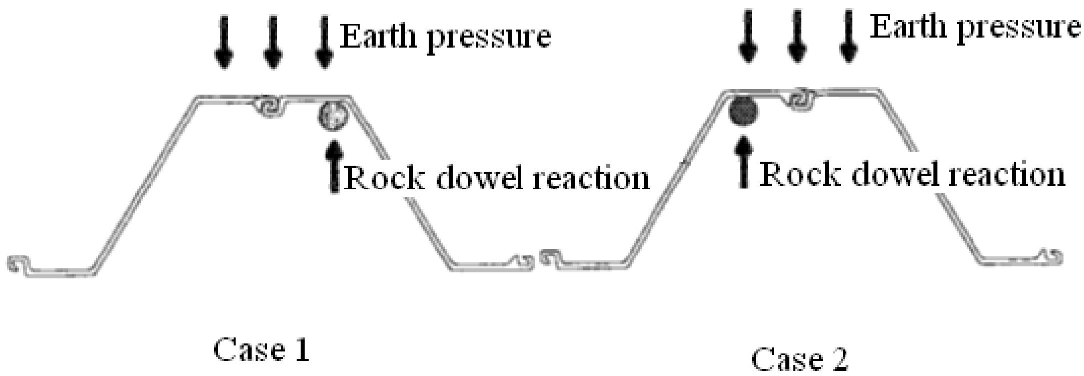


Figure 11. Location of the rock dowel in relation to the interlock of an AZ profile: Case 1: preferred location, Case 2: location conducive to declutching of the interlocks (Kort, Karlsrud, 2008)

2.6 Welded rock dowels

A third method for the installation of rock dowels was mentioned by T. G. Jensen, 2008 in “Sikring av spuntfot” whereby the rock dowel is welded directly to the sheet pile profile prior to driving. In Scandinavia this method has been used in practice. However its use is not recommended in theory as the connection between the rock dowel and the rock is deemed rather unreliable.

The rock dowel is welded into a slot cut into the web of the sheet pile prior to driving as shown in Figure 12. The weld is designed to transfer the axial forces and shear forces at the toe of the sheet pile to the rock dowels, and to withstand the driving forces during pile driving. The rock dowel should protrude a maximum of 100 – 150 mm below the toe of the wall to avoid excessive bending and deformation of the rock dowel while driving into the bedrock.

Each sheet pile profile is fitted with a rock dowel. The sheet pile steel around the dowel is reinforced with welded steel plates also shown in Figure 12. Additionally the corners of the sheet pile profile may be chamfered in order to avoid yielding of the sheet pile steel at the toe while driving against the bedrock.

The sheet pile is careful driven until the rock dowel is embedded in the rock. This method is only suitable where the bedrock is sufficiently soft that it may be penetrated by the rock dowel. An accurate rock model and driving record are required to estimate the degree to which the rock dowel has been embedded in the rock prior to excavation. Due to the uncertainty involved in this estimation, the excavation must be carried out carefully, excavating in small sections and the securing the connection at the toe with inclined toe bolts before proceeding to the excavation of the next section.

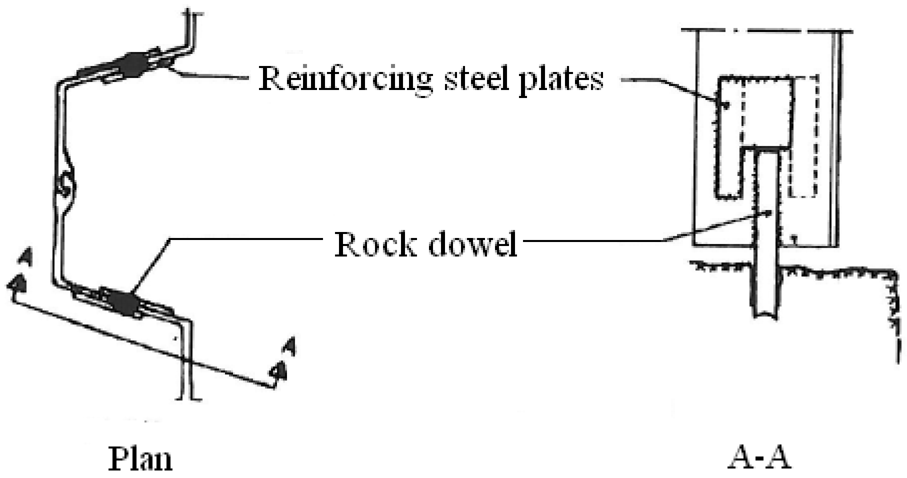


Figure 12. Rock dowel welded to sheet pile. (Jensen, 2008)

Rock dowels welded directly to the wall can be considered fully fixed to the wall, giving full rotational stiffness at the top of the dowel. The connection of the rock dowel in the bedrock may only be assumed to be pinned due to the short, ungrouted embedment of the rock dowel in the rock. Thus the rock dowels conform to the “free” model given in Figure 7. They are designed according to the design procedure for plastic bending set out in section 2.3.2., keeping in mind that axial loads are likely to be present in the dowel if it has not been fully embedded in the rock. In this case the gap remaining between the toe of the sheet pile and the bedrock prevent the transfer of axial forces directly from the wall to the rock.

2.7 Other design considerations

Certain other aspects of the design of the retaining wall can also influence the conditions at the toe and hence the lateral support required.

The anchor system may be designed to minimize the horizontal force at the toe by locating the anchors and adjusting the number of anchor levels so that nearly all of the loads are carried by the anchors. Similar considerations can be applied to the design of the bracing system. As anchors can be designed and installed more accurately than rock dowels, this provides an overall safer and more reliable design. (Suomen Rakennusinsinöörien Liitto. 1974)

If the wall is to be supported by more than one row of anchors, the portion of the sheet pile below the lowest level of anchors may be designed as a cantilever so that support need not be supplied for example by rock dowels or inclined toe bolts. Then the wall must be designed to have sufficient stiffness to carry the horizontal loads. Support of the vertical loads must also be provided either by bracing or by sufficient contact between the toe of the wall and the rock surface so that the loads are transferred directly to the bedrock. (Suomen Rakennusinsinöörien Liitto. 1986)

2.8 Evaluation of the connection

A complete driving record is to be kept during the execution of the piling work. The length of the sheet pile section, the elevation of the top of the sheet pile, and the elevation of the sheet pile toe on reaching the rock and, if it is possible to drive the sheet pile toe into the rock, the final elevation of the toe in the rock are recorded for each sheet pile profile driven. Where rock dowels are used an accuracy of 40 mm is required for these measurements as the capacity of the slender rock dowel sections are very sensitive to the size of the gap between the sheet pile toe and the bedrock (SFS EN 12063:1999).

The degree of contact between the toe of the sheet pile wall and the rock surface may be tested after the sheet pile has been driven using a pile driving analyzer or by performing loading tests. To implement a loading test a high load is imposed on either the sheet piles or the anchors. This load is to be 1,5 times the design load when the design load are calculated using a global factor of safety and 1,3 times the design load when the design load are calculated using partial factors of safety (Laatunen, 2001). Test drillings can also be specified to ascertain the contact between the sheet pile toe and the bedrock (Rakennustietosäätiö RTS. 2006). However such test drillings are not generally necessary. Drilling of the rock is required for the installation of the rock dowels. The elevation of the rock surface at the location of the dowel can be measured and recorded during the drilling.

If a casing attached to the sheet pile is being used for rock dowel installation then the actual gap at the rock dowel can be measured accurately. The distance between the top of the concrete plug at the base of the casing and the toe of the sheet pile is measured prior to driving. The elevation of the top of the concrete plug in the driven sheet pile is measured. Thus the elevation of the toe of the wall is known. Using the rock surface elevation measured during the drilling of the rock dowel, the gap is calculated. Note that the effective gap used in design should be larger than the distance from the toe of the sheet pile to the rock due to flexibility of the connection within the casing. When using a bored installation tube the gap is estimated using the results of the driving record to predict the elevation of the sheet pile toe at the location of the dowel.

As the piling work continues the measurements described above are drawn up in long sections on site so that actual site conditions are easily compared with that assumed in design. The assumed rock surface elevation is compared with that measured during drilling of the rock dowels. The soundness of the contact between the sheet pile and rock can then be assessed using the actual penetration of the sheet piling on site. The size of the gap existing at the location of the dowel is compared with that assumed in the design. Any significant differences should be reported to the designers immediately so that the capacity of the rock dowels to support the conditions existing on site can be assessed.

If the rock dowels are found to be insufficient it is necessary to provide reinforcements to the toe support. Extra casings attached to the wall prior to driving as shown in Figure 3 allow the installation of extra rock dowels to increase to capacity of the connection at the toe of the sheet pile to the bedrock (SFS EN 12063:1999)). If extra casings have not been put in place, rock dowels may be installed using a bored installation tube.

The design of the bracing system may be changed and additional temporary bracing may be installed to provide greater support close to the toe. (Liber, Stockholm. 1984). If the forces at the toe are expected to be quite large then an additional row of anchors may be installed just above the toe to minimize the load acting of the toe (Jensen, 2008).

2.9 Support on excavating to bedrock

Once the excavation has reached bedrock the final lateral support required at the toe can be determined and installed. Typically inclined toe bolts and a reinforced concrete toe beam as illustrated in Figure 13 are installed to either partially or wholly replace the rock dowels as the lateral support at the toe (Liber, Stockholm. 1984).

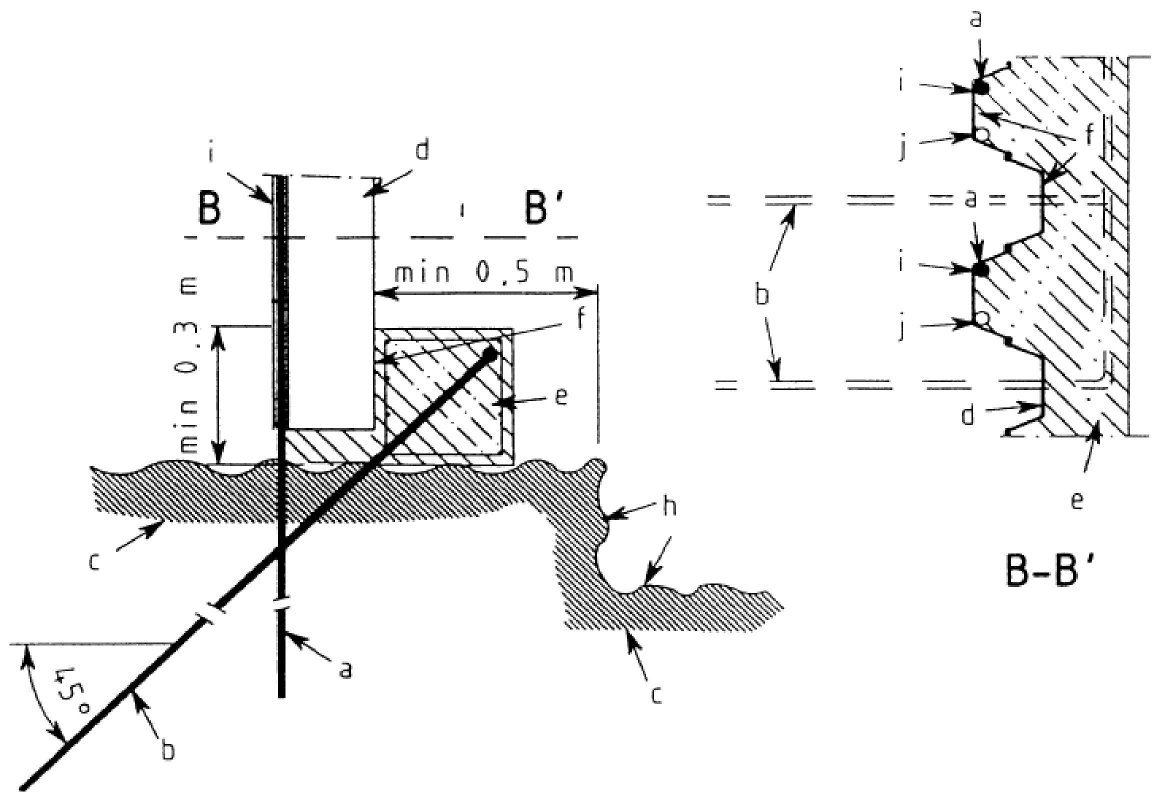


Figure 13. Details of inclined toe bolt and concrete toe beam: a = rock dowel, b = inclined toe bolt bent in the longitudinal direction of the toe beam, c = bedrock, d = sheet pile, e = reinforced concrete beam, f =cleaned surface, h =excavation contour in the rock, i = casing placed where the distance from the sheet pile to the rock is expected to be the smallest, j = spare casing. (SFS EN 12063:1999)

Inclined toe bolts consist of 25-32 mm diameter ribbed steel bars. They must be corrosion protected if the retaining wall is to be a permanent structure. Inclined toe bolts are placed at least at every second profile. They are drilled into the rock at a 45 angle and grouted in place. The length of the toe bolt in the rock is at least 0,5m in good quality rock, though a length of up to 4,5 m may be necessary in poor quality rock. (Laatunen, 2001) The direction of the toe bolt in the rock is chosen taking into account the structure and the dominate fractures sets in the bedrock as well as the slope of the bedrock surface (Liber, Stockholm. 1984).

The portion of the toe bolt extending above the rock is bent to the horizontal and parallel to the wall. It is cast inside the toe beam as shown in Figure 13 and extends at least 0,5 m in the longitudinal direction of the toe beam (Suomen Rakennusinsinöörien Liitto. 1974). Alternatively the portion of the toe bolt extending above the rock is bent to the vertical and welded directly to the sheet pile profile over a minimum length of 0,5m as shown in Figure 14. If conditions allow, toe bolts welded to the sheet pile may be used instead of rock dowels. In this case the bolts are installed and fixed to the wall immediately on exposure of the rock surface. Displacements of 10-30 mm at the toe are common in this situation. (Laatunen, 2001)

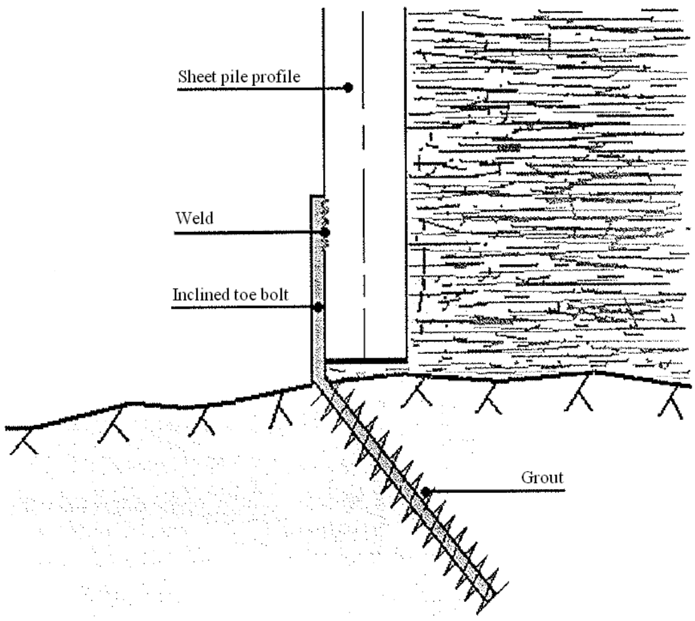


Figure 14. Inclined toe bolt welded to sheet pile profile (Laatunen, 2001)

Inclined toe bolts are designed to resist shear only. This is a valid assumption as the toe bolts are installed after the excavation is complete. At this point it is possible to infill any gap between the toe of the retaining wall and the bedrock by installing a concrete toe beam or by welding steel plates to the toe of the wall to cover the gap.

In Finland the design method laid out in Pohjarakennus RIL 95 (Suomen Rakennusinsinöörien Liitto. 1974) and shown in section 2.3.2, equation (2) is applicable to inclined toe bolts. In this case the first part of equation (2) relates to the capacity of the grout anchoring the toe bolt into the rock or the capacity of the concrete of the toe beam, whichever has the lower cubic strength. Alternatively the shear capacity of inclined bolts may be calculated according to equation (11) given in section 2.3.2, using a partial factor of safety of 1,2 for deep excavations and the material partial safety factor of 1,6 – 2,0 as before. (Laatunen, 2001) These design procedures require that the grouting is properly carried out and the rock is of adequate quality to carry the loads from the toe bolts.

Inclined toe bots are also in tension. The steel capacity is generally considered sufficient to carry the tensile loads. The grout is considered critical in this case and is specified to resist pull-out. (Laatunen, 2001)

After the inclined toe bolts have been installed a reinforced concrete toe beam may be cast on the rock surface against the toe of the wall as shown in Figure 13. The toe beam is at least 300 mm high but should be high enough to cover the gap between the toe of the wall and the rock. It should be remembered that casting a toe beam at a highly variable rock surface requires more complicated form work and bar bending. The toe beam is designed as a continuous reinforced concrete beam, carrying the horizontal toe loads and restrained horizontally at the points of the inclined toe bolts anchored in the rock (Laatunen, 2001).

If the rock at the base of the excavation is to be blasted then the rock dowels cannot be assumed capable of carrying the loads. The rock dowels are not designed to resist blasting vibrations. Moreover the fracture zone resulting from blasting activity decreases the integrity of the anchorage of the dowel in the rock and failure due to pull through on the vertical blasted rock face is possible. Toe support is instead provided by inclined toe bolts cast inside a reinforced concrete toe beam installed prior to blasting (SFS EN 12063:1999). In this case the inclined toe bolts and toe beam are designed to wholly take the shear forces carried by the rock dowels and the blasting vibrations to ensure vertical and horizontal stability during and after blasting. The inclined bolts are anchored deep enough into the rock to extend past the predicted fracture zone due to blasting. They both distribute the horizontal loads from the wall to the undisturbed rock outside the fracture zone and reinforce the fractured rock (Laatunen, 2001). Injection grouting of the rock after blasting can be used further improve the quality of the rock. (Suomen Rakennusinsinöörien Liitto. 1989).

The sheet pile must be located so that the face of the sheet pile on the excavation side is at least 0,5m outside of the proposed blast line. This allows adequate space for the installation of the toe beam. (Suomen Rakennusinsinöörien Liitto. 1989).

3 Lateral toe support: Practice

3.1 Ground Investigation

A good and thorough site investigation prior to the commencement of the work is essential for the design of a suitable toe support and for predicting possible problems relating to installation. The site investigation must take into account both requirements of the design engineer as well as information required by the contractor for the successful execution of piling work on site.

Firstly the aim of an investigation is to map the rock surface as accurately as possible. An accurate measurement of the elevation and contours of the rock surface along the line of the retaining wall is necessary to estimate the required the depth of penetration for the sheet piles, and also to determine chamfering required at the sheet pile toe and the best location of the rock dowels along the sheet pile section. It is also necessary to determine the slope of the rock surface perpendicular to the line of the wall. An unfavorable rock slope, sloping into the excavation, will increase the risk of lateral displacement at the toe, while a favorable one can essentially remove that risk completely. The rock surface is mapped using rock drillings at 5 m to 10 m centers along the line of the proposed retaining wall, with a tighter spacing used for irregular rock surfaces. Only occasional drilling need be taken in the perpendicular direction to assess whether the rock is sloping into the excavation. The drillings extend at least 3m below the rock surface and the resistance of the rock to drilling is measured to ensure that the drilling has reached bedrock and not merely a boulder.

The properties of the rock should also be measured. The friction coefficient of the rock surface indicates the resistance to lateral displacements at the toe due to friction between the toe and the rock surface. The hardness of the rock is important for pile driving. For a heavily broken rock surface and a soft rock type it may be possible to drive the retaining wall into the rock to some extent. Measurements of the extent of fracturing and mapping of the dominant fracture sets in the rock body are necessary for the design of inclined toe bolts and rock bolts.

The drivability of a sheet pile in the soil must also be investigated at this stage, as is highlighted in Eurocode SFS EN 12063:1999. Problems for pile driving can occur in rocky and stony soils as well as in tight soil layers. The investigation should indicate these problems so that suitable driving methods can be determined in advance of the piling work. In addition the likelihood that the sheet pile will be driven to a sufficient depth to make good rock can also be estimated and taken into account when designing the lateral toe support and assessing the suitability of rock dowels.

The necessary properties of the soil for considering the drivability of a sheet pile can be assessed with a combination of weight sounding tests and dynamic probe tests, located at the same points as the drilling tests. Test piling may be carried out in the second phase of the site investigation in addition to supplementary soundings and test pits if necessary. These should be focused in the areas which are deemed to be the most problematic according to the results of the first phase of the site investigation and should provide the information for the design of special measures to improve the drivability in these areas. In doing so, solutions for problems in sheet pile driving can be incorporated in the design at an early stage.

3.2 Driving of sheet piling

In Finland it can difficult to ensure that a sheet pile is fully driven to bedrock due to the prevalence of glacial till deposits usually present as a thick layer up to a few meters deep overlying the bedrock. These predominantly sandy or silty glacial tills often contain stones or cobbles. In addition this till layer was formed under high vertical loads from the overlying glacial ice which compressed the glacial till material into a tight layer. Both of these characteristics give rise to problems during driving. Attempts at driving through the tight till layer may not be successful, decreasing the likelihood that the sheet pile will be driven sufficiently deep to secure contact with the rock. High forces are required for piling through the tight material. These high forces, coupled with the presence of stones and cobbles increases the risk of damage to the toe of a sheet pile and the casing attached to the wall and declutching of the interlocks.

The use of a combination of pile driving techniques can help achieve a better penetration. Hammer driving techniques achieve good penetration in stiff soils. However, if large stones are present this technique can easily lead to damage of the piling and casing as the pile will only be driven directly against any stone it encounters. Vibration driving techniques provides better results in such cases as the vibration can dislodge stones beneath the toe of the sheet pile. As Finnish glacial tills commonly consist of a mix of both stiff soils and stones, better penetration can be achieved by combining these two driving techniques.

To improve penetration in particularly difficult tight or stony soils, the soil may also be predrilled. A row of vertical holes are sunk through the difficult soil layer along the line of the proposed retaining wall. The drilling may be carried out using an auger to depths of 5 - 10 m. The diameter of holes may be less than the width of the wall. Usually holes of diameter 320 – 400 mm are used. The holes may be filled after drilling and prior to wall installation to avoid collapse. Alternatively the soil may be allowed to collapse so that the soil is thus loosened in the region of the holes leaving it possible to drive the piling to the desired penetration level.

The predrilling method has proven very effective in practice. However it is not common due to the time involved and difficulty in accurately positioning the drillings along a line following the undulating profile of wall when the errors in drilling can be up to 0,5m. It is also adds greatly to the cost of piling and so is not used unless there is sufficient reason to do so. Then it may be used only at specific locations on site when it is evident that a particular sheet pile has not reached the desired depth. In this case the sheet pile in question is removed, pre-drilling is carried out and the sheet pile is driven anew.

Finnish bedrock mainly consists of very hard and intact granitic rock. Attempts to drive the sheet pile in this rock would only result in considerable damage to the piling. While the softer rock types such as might be found in Central Europe and highly fractured rock surfaces may be penetrated by the driving forces used for piling, this technique is considered completely impractical for the Finnish bedrock and therefore it is not used.

3.3 Rock dowels

3.3.1 Installation: casing attached to the wall versus bored installation tube

Rock dowels are used widely in practice in Finland and they are installed using either a casing attached to the wall or a bored installation. The use of rock dowels welded directly to the sheet pile is unheard-of. There are differing opinions on which method is the most viable, with the bored installation tube appearing to be more popular.

It is widely recognized that the installation of rock dowels through casings attached to the wall gives the most accurate positioning of the rock dowel and a higher load transfer capacity. However this method is generally not favored due the problem of damage caused to the casing during driving. It has been a common experience that, in attempting the drive the wall to bedrock, the thin walled steel casing has been crushed and distorted to the extent that the installation of rock dowels has been rendered impossible. In these cases it has been necessary to resort to the use a bored installation tube for the installation of the rock dowels.

On the other hand it has been the experience of some of the interviewed experts that casings attached to the wall can be used successfully. It is probable that the soil conditions governed the success of the method in such cases. Thus the suitability of this method must be assessed for each project based on the particular site conditions.

A second factor contributing to the unpopularity of the use of casings attached to the wall is the additional handling involved in welding the casings to the wall. This makes the preparation of the sheet piles more time-consuming and expensive. The casings must again be removed after the extraction of the sheet piles so that the sheet piles may be stored properly before reuse. Furthermore the casings are fixed in place during the working life of the retaining wall and so cannot be reused frequently. In comparison bored installation tubes are removed and reused as soon as each rock dowel is in place.

The use of bored installation tubes is widely preferred, with some considering this to be the only viable rock dowel installation method. Typically the installation tube consists of a drilling tube, usually 80 – 100 mm in diameter though sometimes up to 150 mm in diameter, with a ring bit at the base which is easily capable of drilling though stiff soils.

However, experience has found that this method can give poor results due inaccuracies in positioning the rock dowel. In the design a space of up to 20 mm between face of the wall and tube is generally assumed, allowing minimal displacement to occur at the toe. In reality errors in boring can be quite large due to the slenderness of the installation tubes which deflect easily in stiff soils or on contact with stones and cobbles. Typically boring errors are in the region of 100 mm. But these may be much greater depending on the site conditions, sometimes resulting in a space between the wall and the tube of up to 500 mm as can be seen in Figure 15. In this case there is effectively no lateral support at the toe of the wall. Excessive displacements of the toe may then occur, leading to possible progressive failure due to the resulting increase of the loads on the anchors.

Figure 15. Inaccurately placed rock dowels using bored installation tubes. (Photo: Tapio Ranta-Aho)

Secondly, it is understood that the lack of fixity at the top of the dowel greatly restricts the load transfer capacity of the connection at the toe. This is highlighted in the previous chapter.

3.3.2 Installation and design of rock dowels

Bored installation tubes are commonly bored from the existing ground level, as typically requested by the designers. It is also possible to bore these tubes from a lower excavated level. This level can be determined by calculating the minimum depth of penetration required for overall stability without providing extra restraint at the toe. It must also be noted that, if the toe is unrestrained, displacements of perhaps a few centimeters will develop at the toe as the excavation proceeds. These displacements and the resulting increased loads on the anchors must be considered when deciding the excavated level from which the installation tubes may safely be bored. The benefit of boring from a lower excavated level is the shorter boring length to reach bedrock and hence a cheaper installation process.

Similar to that described in literature, grade S355 circular steel bars of 50 – 100 mm diameter are commonly specified by designers. These are mainly designed for shear only using the equations set out in Pohjarakennus RIL 95 as described in section 2.3.2, equation (2) and including a large factor of safety to account for any gaps below the toe of the wall. Because the drilling is the most expensive component, specifying bigger bars at wider spacing results in a more economical design. Contractors often prefer to use the smaller bars, particularly below 60 mm diameter as the equipment for drilling smaller bars is lighter and usually more readably available. Bars as small as 32 mm diameter, have been used in practice with no ensuing stability problems for the excavation. However, due to the extremely low capacity of these bars, the stability in this case could not be accounted for by traditional theoretical design. It is highly likely in these cases that sound contact had been achieved between the sheet pile and rock such that bending of the dowels was minimal and lateral restraint was improved by the friction between the tip of the wall and the rock surface.

Rock dowels located in a casing attached to the wall are not necessarily grouted into the tube. Instead they may be extended a sufficient length, approximately 1,5 – 2 m, into the casing to ensure they are secure within the casing

Rock dowels installed through a bored installation tube can be connected to wall in a number of ways once the excavation has reached the level of the toe of the wall. Dowels sufficiently close to the wall may be welded directly to the wall. They can then be considered to have full rotational stiffness at both ends and obtain a corresponding improvement in their load transfer capacity. However the weld increases the difficulty in removing the sheet pile after their use. If a rock dowel too far to be welded directly to the wall, steel plates may be packed between the dowel and the wall and welded in place to ensure restraint of the toe. Commonly though, the dowels are not fixed to the wall and may be simply bent back towards the wall by the digger bucket during excavation to reduce the space between the wall and the dowel.

3.4 Friction

Where the sheet pile profile has been driven fully to bedrock, any vertical axial loads carried by the sheet pile are transferred directly to the rock at the toe. These vertical loads give rise to friction between the base of the sheet pile profile and the rock providing a means of shear force transfer for the horizontal toe loads as shown in Figure 16.

The friction force between the base of the sheet pile profile and the rock is calculated by:

$$F_f = \mu F_v \tag{21}$$

where F_f is the friction force along the toe [kN/m]

μ is the coefficient of friction between the toe and the rock surface [-]

F_v is the vertical axial force at the toe [kN/m]

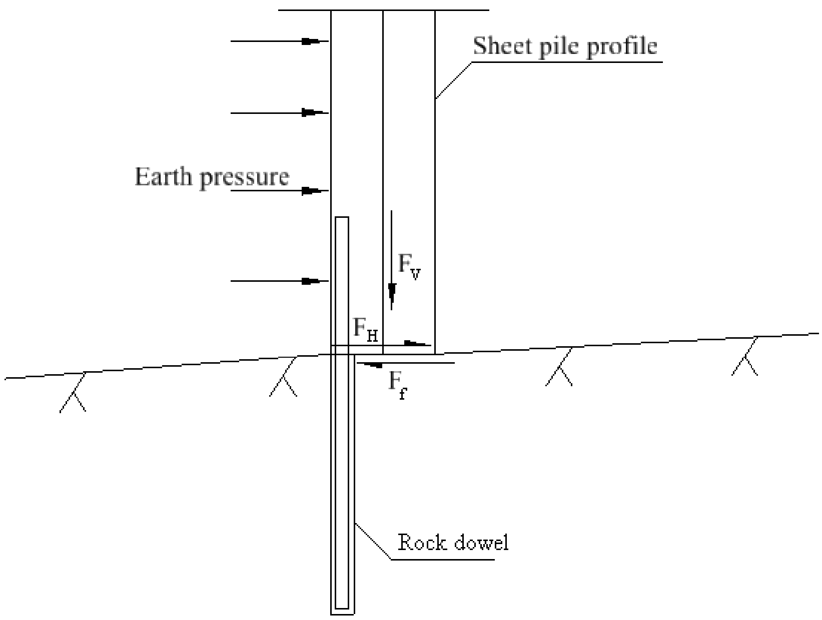


Figure 16. Section through sheet pile profile indicating friction acting across the base

The above equation predicts that under large vertical axial loads the shear transfer capacity due to friction is very significant. The vertical axial forces may be calculated accurately from the vertical component of inclined anchor loads. However the value of the coefficient of friction is rather difficult to determine accurately. It can be estimated from rock core sample. However, finding a good representative value is difficult and any value for the coefficient of friction measured from core samples should be used with caution. Therefore, while it is recognized that that friction very likely contributes significantly to lateral restraint at the toe, it is considered too risky to rely on in design.

It is common to specify rock dowels with sufficient capacity for the design shear loads and consider friction as a reserve capacity only. On rare occasions where the conditions allowed and where the installation of other toe support systems is impossible, a limited friction force may be permitted in design. Then it must be confirmed that there is sound contact between the sheet pile and the rock and that the rock surface slopes up towards the excavation such as to resist lateral displacement of the toe. The capacity of the toe steel to transfer the loads locally must also be assessed. It may be necessary to reinforce the toe with welded steel plates. This reinforcement also helps to prevent crushing of the toe steel on driving against rock. Friction must not be considered if the rock surface is sloping into the excavation or if there is no contact between the wall and the rock over a distance of several meters.

3.5 Alternative solutions

If the solid circular section of the rock dowels cannot provide the capacity required, drilled piles are installed instead. Their higher shear and bending capacity allow them to take both higher horizontal loads and higher bending moments due to large gaps. Drilled piles have been used successfully to provide lateral toe support at gaps of up to one meter. They are however considerably more expensive than rock dowels, both in material and installation costs.

Drilled piles consist of 100 – 300 mm diameter hollow circular sections. They are drilled either from the ground surface or from a suitable lower excavated level directly into the rock such that they are located immediately in front of the toe of the wall similar to rock dowels. Since drilled pile sections are much larger and sturdier than rock dowels, they do not deflect during drilling and so can be very accurately placed in the correct location. Depending on rock quality, the piles are drilled 1 – 2 m into the rock; longer embedment lengths are necessary in weaker or poor quality rock conditions. The drilled piles are not necessarily grouted in place as the long embedment and the side friction against wall of the drill-hole prevent pull-out of the drilled pile.

Drilled piles are designed as cantilevers, anchored at the level of fixity in the rock and carrying the horizontal toe loads which are assumed to act as a point load at the level of the toe of the wall. Their capacity may be increased after excavation by welding the drilled pile directly to the wall. A composite section consisting of an I-section placed within the drilled pile section and infilled with concrete can offer an even stiffer toe restraint.

Sheet pile walls supported by rock dowels are the most commonly used retaining wall system due to their relatively low cost and quick installation. However difficult soil conditions such as stiff or stony stones can make this method unsuitable. Large boulder overlying bedrock will prohibit the use of rock dowels as the boulders do not necessarily provide secure anchorage for the rock dowel as can be seen in Figure 17. The piles may also be installed as a means of providing reinforcement to the lateral toe support where rock dowels have been installed as the primary means of lateral toe support but have been found to be insufficient.

Figure 17. Rock dowel installed through a boulder. Photo: Tapio Ranta-Aho

In such cases drilled pile walls may prove more viable. These walls consist of large interlocking circular steel pile sections, approximately 600 mm diameter. They can be drilled directly through stiff soils, boulders and weak rock formations to attain secure anchorage in good quality intact rock. Reliable support at the base of the wall provided by the deep embedment of the wall itself in bedrock with lateral loads transferred directly to bedrock by side pressure along the length of the drilled pile wall embedded in the rock. Combi-walls consist of a composite of drilled pile sections and sheet pile section which spans laterally between drilled piles. These walls offer similar benefits to drilled pile wall.

3.6 Excavation and final support

It is commonly considered that all problems relating to the inaccurate positioning of rock dowels can avoided by carrying out the lowest part of the excavation carefully in a step-wise fashion. A small width of roughly 2 m is excavated such that the overall stability of the retaining wall is not compromised. The integrity of the connection at the toe is assessed and ensured for each section, applying reinforcements as described in the section 2.6 if necessary, before excavating the next section. This is especially relevant for rock dowels installed through a bored installation tube. In fact it is generally considered most efficient to use a bored installation tube in combination with a step-wise excavation program as it has often been found that only a limited number of dowels would be inaccurately placed and in need of reinforcement. This step-wise excavation can be carried out while other sections of the wall are being driven and so it does not result in delays to the timetable.

Due to the low cost and the seepage control provided, inclined toe bolts and a reinforced concrete toe beam are generally installed after excavation even when no blasting is to be carried out and the rock dowels are providing adequate support. Dowels which have been incorrectly placed too far from the wall can be cast inside this beam so that their shear capacity can be used in the final state.

As has been mentioned an inadequate toe support will manifest as movements of the entire wall as the excavation proceeds. A standard monitoring system measuring displacements at various points along the height of the wall may then also indicate the performance of the toe support.

Figure 18. Concrete toe beam along the toe of a sheet pile wall Photo:Iikka Kärki

If the rock dowels have been properly installed the toe beam may be cast as a whole on competition of the entire excavation. It is more common that the toe beam is cast in short sections immediately on uncovering the toe as the excavation proceeds in a step-wise fashion.

4 Sealing at the toe: Theory

4.1 Groundwater exclusion in excavations

The work environment inside the excavation must be kept dry. Where lowering of the groundwater in the area surrounding the excavation cannot be allowed a groundwater exclusion system around the periphery of the excavation must be installed. This minimizes or prevents the ingress of groundwater into the excavation and minimizes disturbance to the groundwater surrounding the excavation. For excavations extending to bedrock, in addition to the usual sealing requirements for the retaining wall itself, sealing of the interface between the toe of the retaining wall and bedrock, as well as sealing of the exposed surface of the bedrock must be carried out.

4.2 Toe beam

The installation of the concrete toe beam is most widely recommended method for the control of seepage beneath the toe of a retaining wall (Suomen Rakennusinsinöörien Liitto. 1989). Thus the toe beam is often installed even where no blasting is carried out and rock dowels provide sufficient lateral toe support. When the main purpose of the toe beam is the prevention of seepage it must be cast as soon as possible on reaching the bedrock. The concrete of the toe beam also provides corrosion protection for the steel at the toe of a sheet pile. Corrosion protection is a requirement for permanent retaining walls. If grouting is to be carried out behind the toe of the wall after excavation, the toe beam can provide sufficient restraint to the toe of the wall against the pressures introduced by the grouting work (Jensen, 2008).

4.3 Grouting

4.3.1 Overview

A soil layer of high permeability at or below the level of the toe and fractures in the bedrock at the base of the excavation provide a pathway for groundwater flow. Where the flow is high, grouting of these pathways is necessary in order to decrease or prevent groundwater ingress into the excavation (Virkkunen, 2010). Remedial grouting may be carried out after excavation if the conditions allow and groundwater flow is not too high. Generally it is preferable that grouting is carried out before excavation as this ensures a watertight structure immediately on excavation.

A number of grouting techniques are available. Penetration grouting and jet grouting are the most popular techniques used in operations to control groundwater flow in soil and rock.

4.3.2 Penetration grouting

Due to the low soil strengthening provided by this technique, penetration grouting is used mainly for groundwater control. A borehole is drilled into the soil or rock to the required depth. Grout of low viscosity is then injected into the soil or rock under low pressure. The grout replaces any water or air present in the voids within the soil or rock mass so that the voids are filled with the grout. Some fine grained material may also be displaced in this process. The fines and the water are removed from the soil mass by rising up through the annulus of the borehole and the drill string to the ground surface. The procedure is outlined in Figure 19.

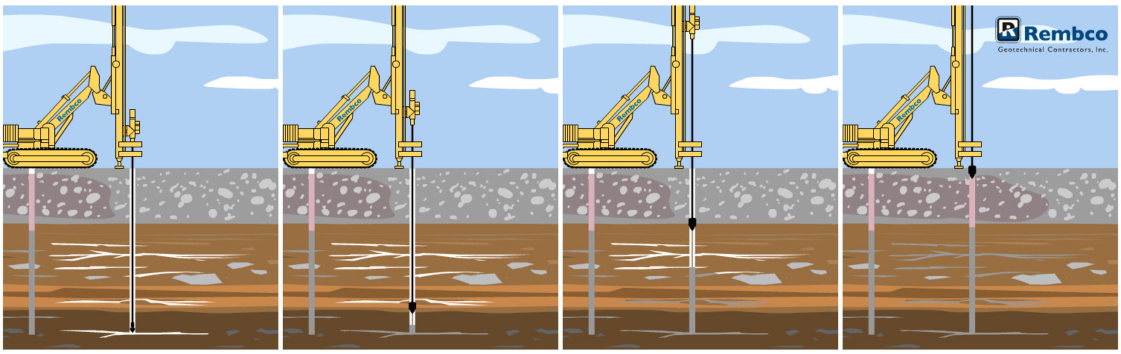


Figure 19. Procedure for penetration grouting (Rembco. Accessed 19.11.2011 <www.rembco.com/pressure_grouting.html>)

The pressure and the velocity of the grout injection are high enough to ensure penetration. However the grout injection should neither cause excessive disturbance to the soil or rock nor introduce additional voids due to fracturing of the soil or rock. Thus the basic soil structure remains unchanged. (Kazemian, Huat, 2009). The injection pressure should not exceed the overburden pressure.

Grouts installed by this method cannot adequately penetrate soils where the permeability is less than $1 \cdot 10^{-6}$ cm/s such as are typical in silts and clays and dense soils. In such soil the large grout particles cannot easily pass through the small voids present in the soils. Thus penetration grouting is generally only suitable for use in coarse grained soils and for sealing fracture and other openings occurring in rock. (Techtracker. 2007)

4.3.3 Jet grouting

Jet grouting is a more recently developed technique suitable for a wider range of applications and soil conditions. It differs from other grouting techniques in that high pressures are used to break up the soil structure completely. The soil is then mixed with the grout to form a solid impermeable homogeneous mass. (Kazemian, Huat, 2009).

A borehole is drilled into the soil or rock to the required depth. As the drill head is being withdrawn from the borehole, it is rotated. Fluids are injected into the soil or rock under high pressure at high velocity from nozzles located just above the drill bit. A column of grouted material is thus produced. An impermeable barrier may then be formed by overlapping grouted columns, as shown in Figure 20.

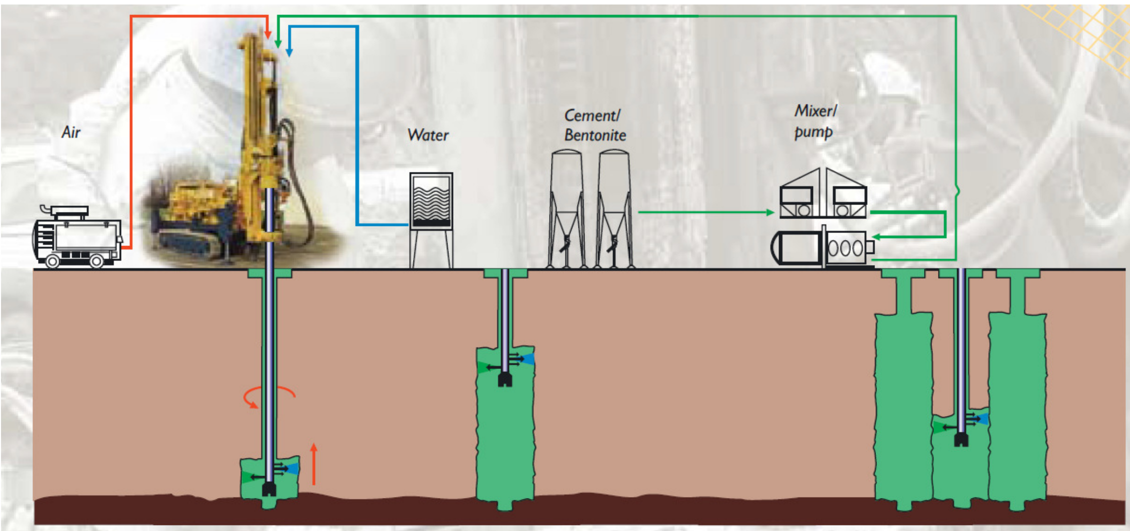


Figure 20. Jet grouting: triple system (Keller. Accessed 19.11.2011 <www.kellerge.com.au/pdf/Grouting_Brochure>)

Three main systems of jet grouting are available. The single system uses the fluid grout to both break up and solidify the soil. The double system utilizes the grout in a similar manner to the single system but a stream of compressed air surrounds the jet of grout to improve grout penetration into the soil. In the triple system a jet of water surrounded by a stream of compressed air is first used to break up the soil and removes some of the fine grained material. The grout is then injected into the disturbed ground, giving even further penetration of the grout into the soil. (SFS EN 12716:2000)

As the structure of the soil is completely broken down, this grouting technique is suitable for any soil types including dense soils of very low permeability. Thus jet grouting offers a grouting solution where penetration grouting is unsuitable.

5 Sealing at the toe: Practice

5.1 Ground investigation

An investigation of the groundwater conditions aims to determine the risk of seepage into the excavation and to locate the main pathways for groundwater flow. In excavations extending to bedrock the interface between the toe of the wall and the bedrock, as well as the bedrock itself must be included in this investigation.

The Finnish glacial till is quite a variable material with a wide range of permeability. Mostly the permeability of the glacial till is low and problems due to excessive seepage through the glacial till layer do not occur. However, occasionally the flow through this layer can be high. In particular the investigation should consider the properties of the 200 mm of soil overlying bedrock. It is possible that the permeability of this thin stratum differs from the rest of the material in the soil layer. The soil of this thin stratum has been affected and modified by the movement of the groundwater over the bedrock. Thin anomalous layers have been found in practice in this narrow region above bedrock, differing greatly from the overlying soil and forming in a layer of very high conductivity through which significant flow occurred.

A system of groundwater monitoring tubes is installed in the area surrounding the proposed excavation in advance of the excavation work. Monitoring of the groundwater levels in this system of tubes determines the direction of groundwater flow and groundwater pressure. Monitoring tubes extending to the layer overlying bedrock are included in this system to detect possible pathways for high groundwater flow in the region of the toe of the proposed retaining wall.

The ability of the soil to accept grout must be considered where it is likely that grouting of the soil will be necessary to control groundwater flow. The choice of suitable grouting techniques and the composition of the grout used are based on the permeability, grain size distribution, void size and the void distribution. The existence of high groundwater seepage makes the use of water-soluble grouts unsuitable.

Consideration of the drivability of sheet piling in the given soil conditions will indicate the possible occurrence of large gaps between the toe of the wall and the bedrock. The extent of sealing required to cover large gaps can then be estimated at an early stage. Weight sounding tests, dynamic probe tests and, if necessary, test piling are carried out to assess drivability as described in Section 3.1.

An understanding of the overall geology of the area can help understand groundwater flow. The intact granites prevalent in Finland generally have a low permeability. Core analysis from within the proposed excavation and along the edge of the proposed excavation will give an estimate of the degree of fracturing in the rock and of the size of the fissures to be filled. Hence the quantity of grout required and extent to which the grouting must be applied to control the flow of groundwater through the bedrock can be estimated.

A good site investigation is necessary for successful design and implementation of a groundwater exclusion system around the periphery of the excavation in areas where lowering of the ground water level is not permitted. The successful installation of the groundwater exclusion system installed prior of excavation is important as it may be difficult to install remedial measures after the excavation has been completed and flow has already begun.

Due to the variability of soil, previous grouting experience in the area and in similar soil conditions, in addition to an understanding of the principles behind the grouting techniques, are invaluable to the successful design and installation of grouting.

5.2 Seepage beneath the toe

Sealing against the ingress of groundwater and fine grained material into the excavation is often a more common and more difficult practical problem than stability in deep excavations extending below the groundwater level.

Where complete driving of the sheet pile to the rock level has not been achieved, the resulting gap between the toe of the wall and the bedrock creates an open face across which groundwater may flow. High driving forces at the toe while driving against bedrock can cause declutching of the interlocks, creating openings along the wall. These may provide a pathway for groundwater to enter the excavation. Careful driving of the wall to full contact with the bedrock using suitable driving techniques is important for minimizing problems leading to an increased ingress of groundwater into the excavation. Driving the sheet pile toe into a channel blasted into the rock further aids the prevention of groundwater flow beneath the toe.

The presence of a thick glacial till layer overlying bedrock common in Finland often makes driving of the sheet pile to full contact with the bedrock difficult to achieve. However this glacial till predominantly has a low permeability. Significant flow will generally not occur through this layer if gaps below the toe are sealed immediately on exposure as the excavation proceeds. Gaps between the toe of the sheet pile and the bedrock and openings along the wall are commonly patched with welded steel plates. Where the gap below the toe of the sheet pile is large a layer of shotcrete is applied over the excavate soil face at the gap and covering the rock dowels, as shown in Figure 21. In higher groundwater flow the application of polyurethane injection to the open soil face may be appropriate. This synthetic compound reacts rapidly on contact with water. Therefore it is not easily flushed away and sets to provide a watertight seal even in high groundwater flow.

Figure 21. Shotcrete used to close the gap below the sheet pile toe and the rock (Photo: Tapio Ranta-aho)

5.3 Toe beam

As the toe beam is constructed after the excavation has reached bedrock, seepage beneath the toe will have already occurred before the toe beam is fully cast. This flow is generally not significant if the toe is driven into glacial till of low permeability overlying bedrock. Before construction of the toe beam, a drain is installed at the toe of the wall which channels the small quantity of inflowing water to the main drainage system, allowing the toe beam to be cast. Once the toe beam is complete, this drain is injected and sealed with grout so that a watertight seal is formed along the toe. If the flow under the toe is quite high however, this type of drain is insufficient and it is necessary to decrease the flow for example by employing grouting techniques before the toe beam can be cast.

5.4 Grouting

5.4.1 Penetration grouting

Grouting of the soil may be carried out to control the groundwater flow so that the toe beam may be cast. Alternatively grouting may completely replace toe beam as the primary means seepage control at the toe of the wall.

Penetration grouting is cheaper to install than jet grouting and is therefore often preferred for costing reasons. However it has been found that this technique gives poor results in a number of soil conditions prevalent in Finland. The level of success of this method in the Finnish glacial has been variable, depending on the particular composition of the glacial till present on site. In general penetration grouting is not very effective in Finnish glacial tills due the poor penetration of the grout in compact soil structure of the till. Above the groundwater level the grout can flow by gravity and spread evenly into the voids present in the soil. Below the groundwater level it has been found that it can be difficult to ensure that the grout has been properly distributed in the soil. As a result it is uncertain prior to excavation whether an adequate reduction of the permeability of the soil has been achieved.

Penetration grouting cannot be placed where there is high groundwater flow as the grout is simply washed out of the voids it occupies. If the grout is disturbed in this manner, piping of the grout barrier occurs. Prior to excavation it is impossible to predict the extent of the damage caused to the grout barrier.

Installation of penetration grouting alters the soil in such a manner that the subsequent installation jet grouting is very difficult. Thus the suitability of penetration grouting must be confirmed using the results of the site investigation before any grouting work is carried out. Applying remedial measures to inadequately grouted soil is very difficult and costly.

5.4.2 Jet grouting

Where penetration grouting cannot provide a sufficient solution, jet grouting is the preferred alternative. It can be installed successfully in all soil types making it suitable for the tight Finnish glacial tills overlying bedrock. However the presence of large stones in the glacial till can cause difficulties for the grouting work. Large stones form obstructions to the injected grout and ungrouted regions may remain behind these stones. Increasing the number of grouting application points helps decrease the occurrence of these ungrouted regions in such conditions.

The application of jet grouting to a soil forms of a solid plug of grouted material of high bearing capacity. Where sheet piles have not been driven sufficiently deep to attain full contact with the bedrock, jet grouting may be installed under the toe of the wall and extending down to bedrock. The solid plug of grouted material is capable of transferring the vertical loads from the toe of the wall to the bedrock by compression so that vertical settlements of the wall do not occur.

For a wall supported vertically in this manner, the lateral toe support must be sufficient to prevent horizontal displacements at the toe. If insufficient lateral support is provided outward movement of the toe can occur such that the toe of the wall extends past the edge of the solid plug of grouted material. The wall is then no longer supported vertically by the grouted material and vertical settlements may occur, leading to possible progressive failure of the retaining wall system.

Furthermore, the space formed between the face of the wall and the jet grouting columns behind the wall due to the outward movement of the wall provides a path for seepage. Therefore it is important to ensure the sufficient rock dowels are properly installed. Any space between the retaining wall and the rock dowel discovered on excavation of the toe of the wall must be packed immediately to prevent the outward movement at the toe.

Standard jet grouting techniques cannot be carried out in conditions of high groundwater flow. Therefore jet grouting cannot be used as a remedial measure where excessive seepage has already begun across an excavated soil face. New jet grouting methods are being developed for conditions of high groundwater flow but these are still rather expensive and not commonly used.

Jet grouting is an expensive soil grouting technique. However it is becoming quite popular due to the reliability of this technique in a wide range of soil conditions. Where a watertight seal is crucial jet grouting is deemed the only dependable solution. The use of jet grouting also avoids delays to the timetable due to remedial work.

5.5 Choice of retaining wall

If seepage beneath the toe of the wall is a serious concern, this may dictate the type of retaining wall to be used. In regions of softer bedrock such as those found in Central Europe secant piles may be bored into bedrock to provide both good anchorage and a watertight seal. Boring of secant piles into the hard Finnish bedrock is impossible. A diaphragm walls extending to the bedrock level is more suitable in this case. Diaphragm walls such as slurry walls and secant pile walls can be constructed so that the base of the wall is fully in contact with the surface of the bedrock. All of the soil overlying bedrock is removed. A layer of sealant may be applied to the rock surface beneath the base of the proposed wall. The concrete of the wall is cast flush with the rock surface creating a tight connection between the base of the wall and the bedrock though which only minimal flow will occur.

6 Case study: Urheilupuisto

6.1 Site overview

The Länsimetro project entails the extension of the existing Helsinki metro line under the management of Länsimetro Oy. The extension consists of a 14 km long metro line mainly bored through rock, with stations and service tunnels providing access from the ground level to the track. The geotechnical department of Finnmap Infra Oy, previously Geomap Oy, is currently involved in providing geotechnical design for the deep excavations to bedrock required for the station and service tunnels.

As part of this project the proposed Urheilupuisto metro station and service tunnel are to be built at Jousenpuisto, Espoo. Before work commenced, Urheilupuisto was a greenfield site. The site is bounded on one side by Merituulentie and on another by Koivu-Mankkantie. Approximately 90m from the eastern edge of the proposed excavation are terraced houses. The foundations of these buildings are bearing on soil. More critically the Espoo Sports Center, the foundations of which are also bearing on soil, lies approximately 8m from the northern edge of the proposed excavation. These structures are highly sensitive to settlements due to lowering of groundwater levels and hence place tight restrictions on the disturbance allowed to the groundwater due to the excavation works. A map of the site is shown in Figure 22.

The station will provide access from the street level at an elevation of +3,5 to the platform level which is at an elevation of -20,0. Space will also be provided for an underground multi-story car park. The large excavation necessary for this will be approximately 177 x 68 m. The soil will be excavated down to the bedrock. The bedrock will then be blasted to reach the proposed level of the metro line.

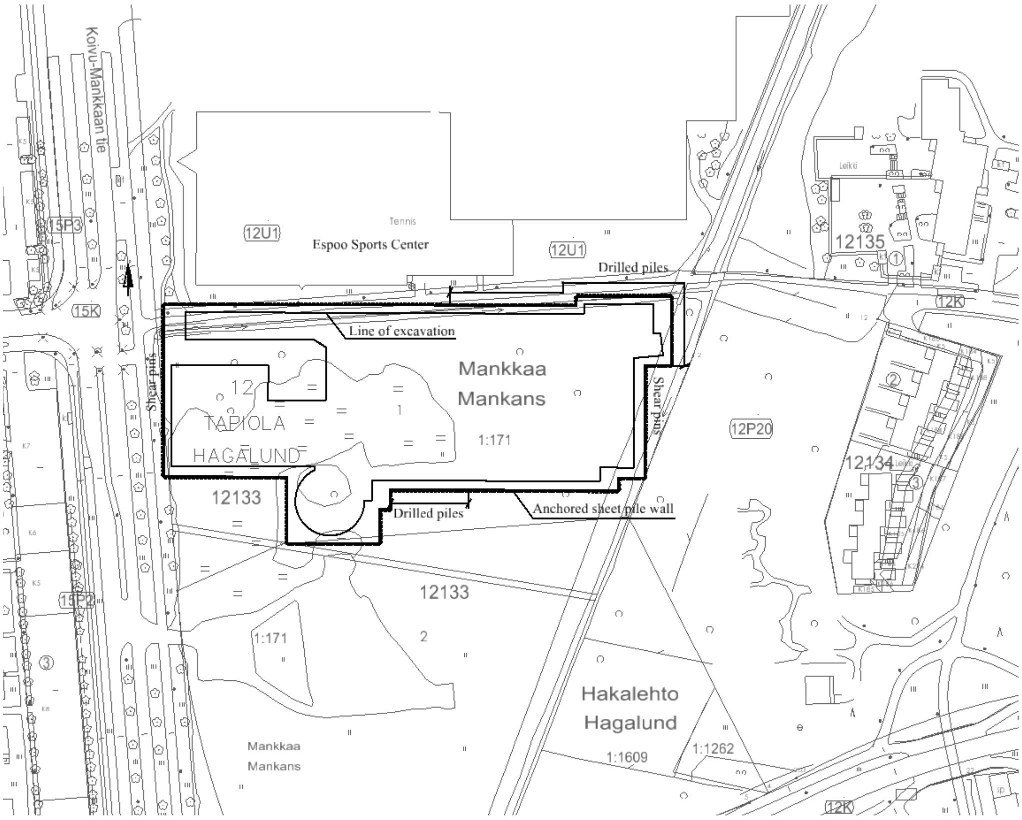


Figure 22. Map of Urheilupuisto site (Finnmap Infra Oy.)

A thorough ground investigation was carried out to provide the necessary data both for the design of the retaining wall and for the ground water management plan.

The ground surface is quite flat and at an elevation of approximately +3,2. Below the thin dry crust lies a very soft and thick organic clay layer. Underneath the clay and overlying the bedrock is a layer of glacial till. This varies in thickness from less than half a meter to more than five meters. Soundings have shown this layer to be quite stiff and it was found to contain some stones and boulders. In general the permeability of this layer was found to be quite low. However one sample taken from the till showed the presence of gravely sand. It is assumed this is an isolated lens but it must be kept in mind that this may also represent a larger conductive layer. The soil profile is shown in Figure 23.

A rock model has been made based on the results of drilling tests taken along or close to the line of the proposed retaining wall. The rock surface level varies between -1,0 and -11,0. The maximum depth of bedrock along the line of the proposed retaining wall is approximately 15m. The rock is found to be highly fractured in places, providing a seepage path for groundwater flow.

The groundwater elevation and the flow in the glacial till layer were measured by installing a number of groundwater monitoring tubes in advance of the excavation work to measure the natural groundwater elevation. The groundwater was found to be quite flat with little variation, +1.5 to +2.5 in the area surrounding the excavation.

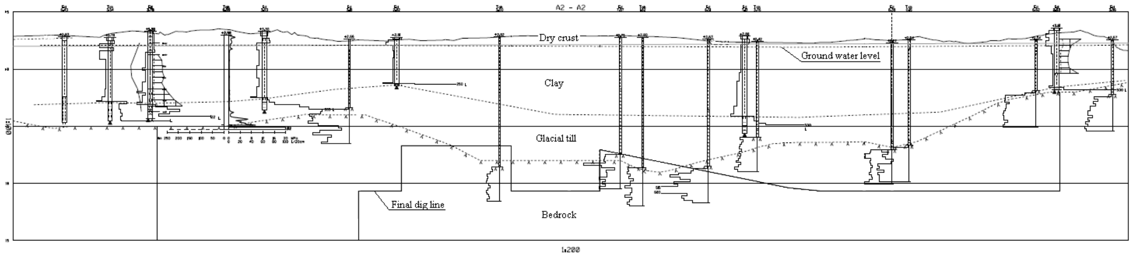


Figure 23. Soil profile and groundwater level at Urheilupuisto site. (Finnmap Infra Oy.)

6.2 Design of lateral toe support

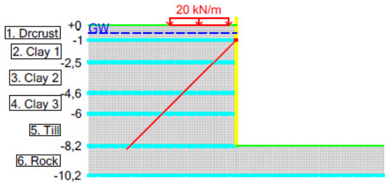
6.2.1 Design Overview

The excavation through the deep soft soil is to be supported by an anchored sheet pile wall consisting of AZ36-700N profiles extending down to bedrock. During the excavation phase rock dowels will mainly be used to provide lateral support to the toe. In the deepest part of the excavation however it has been decided to use drilled piles which both stiffen the sheet pile against the high earth pressures and provide lateral support at the toe. Blasting of the rock at the base of the excavation is to be carried out down to the level of the proposed platform. Therefore final support must be provided by inclined toe bolts and a reinforced concrete toe beam.

6.2.2 Design of rock dowels

An analysis of the sheet pile wall was carried out using Novapoint GeoCalc, a 2-D computer program suitable for the analysis of geotechnical problems. The input data is shown in Figure 24. The latest version of GeoCalc 2.2 allows the horizontal stiffness to be specified for the connection of the toe to the bedrock. As rock dowels will allow some displacements they are not modeled as fully stiff and a suitable horizontal stiffness of 2 kN/m was specified.

The output data yielded in the analysis included the shear force distribution along the sheet pile wall. The shear force distribution is shown in Figure 25 and represents the shear force per meter length of wall. The value of the shear force at the toe of the wall corresponds to the horizontal load imparted by the wall on the rock dowels. As shown in Figure 25 the horizontal force at the toe of the wall is 222 kN/m.



Wall Type	Sheetpile
Name	AZ 36 - 700
Manufacturer	Arcelor
Cross Section Area [m^2]	0,0216
Calculation Width [m]	1
Inertia Modulus [m^4]	0,0008974
Section Modulus [m^3]	0,0036
Elastic Modulus [kPa]	210000000
Flexural Stiffness [kNm^2]	188454
Axial Stiffness [kNm^2/m^2]	4536000

Name	A [mm^2]	L [m]	α [°]	h [m]	F [kN]	Elastic Modulus [kPa]	Overdig [m]	Horizontal distribution [m]
Level 1	2000	10,5	45	1	100	210000000	0	3,6

Id	Layer Name	z [m]	h [m]	γ [kN/m³]	Φ [°]	c [kPa]	Δc [kPa/m]	K0ud	d/ud	Material Model	δya	δyp	ξ50a	ξ50p	m	n	k
1.	Drcrust	0	1	16	0	20	0	1	Undrained	MCM					40	1	1
2.	Clay 1	-1	1,5	16	0	5	0	1	Undrained	MCM					40	1	1
3.	Clay 2	-2,5	2,1	16	0	7	0	1	Undrained	MCM					40	1	1
4.	Clay 3	-4,6	1,4	16	0	11	0	1	Undrained	MCM					40	1	1
5.	Till	-6	2,2	20	34	0	0	0,44	Undrained	MCM					200	0,5	0,5
6.	Rock	-8,2	2	20	45	10	0	0,29	Drained	MCM					10000	1	1

Figure 24. Input data for the analysis of a sheet pile wall using Novapoint Geocalc version 2.2.

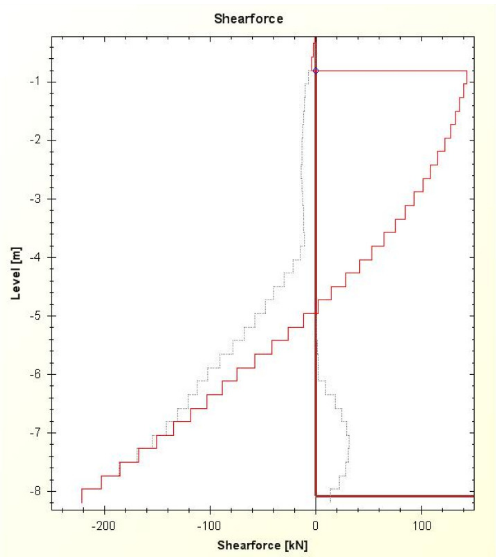


Figure 25.Output data from the analysis of a sheet pile wall using Novapoint Geocalc version 2.2.: shear force distribution along the sheet pile wall

Rock dowels are placed at every second sheet pile profile. The width of each AZ36-700N sheet pile profiles is 700mm. Thus the spacing of the dowels is 1,4m. The horizontal load per dowel given by equation (1):

$$V_{Ed} = q_{Ed} c \qquad \text{(bis 1)}$$

$$= 222 \text{ x } 1,4 = 311 \text{ kN}$$

The rock dowels consist of Grade S355 solid steel bars. The rock dowels are to be installed through a casing attached to the wall, extending at least 0,5 m into the casing. Due to the fractured surface of the bedrock rock dowels are to be drilled at least 1 m into the rock and grouted in place using grade C35/45 grout to ensure full fixity.

The rock dowels were primarily designed according to the procedure set out in Pohjarakennus RIL 95 and as given in equation (2).

$$V_{Rd} = 3 \cdot 10^3 \cdot d^2 \sqrt{10{,}19x10^3 \cdot f_{ck}}$$
$$or \qquad \text{(bis 2)}$$
$$V_{Rd} = \frac{\pi d^2}{20} \cdot \sigma_{ult}$$

A graph of the shear capacity of bars from 50 mm to 100 mm diameter calculated according the equation (2) is shown in Figure 26. From this graph it can be seen that the strength of the grout governs the capacity of the connection. For a horizontal load of 311 kN rock dowels of 70 mm diameter are sufficient.

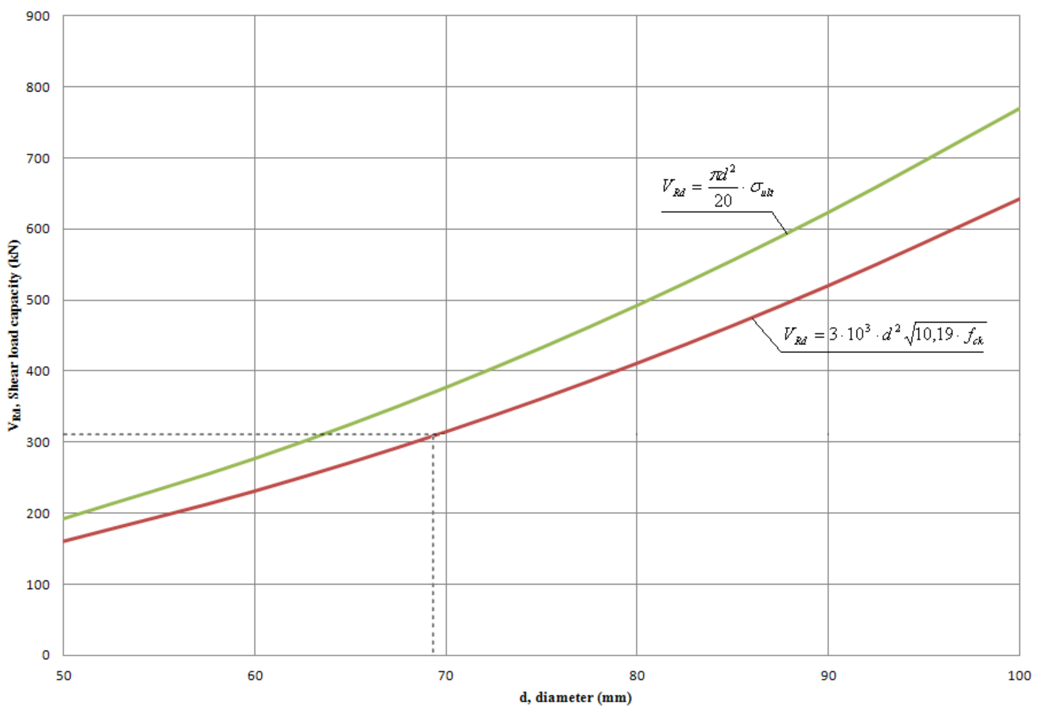


Figure 26. Shear capacity of the rock dowel connection according to Pohjarakennus RIL 95, where $f_{ck} = 45 \times 10^3$ kN/m^2, $\sigma_{ult} = 490 \times 10^3$ kN/m^2.

The capacity of the rock dowels to transfer the horizontal toe loads in bending can be assessed. Rotational stiffness can be assumed at both ends of the dowel due to the installation of the dowels through a casing attached to the wall, and the “fixed” model described in Section 2.3.2 can be applied.

A graph of the horizontal load capacity of bars from 50 mm to 100 mm diameter, shown in Figure 27, is calculated according to the plastic bending capacity given in Equation (9) and limited by the shear capacity given in Equation (11).

$$V_{pl,Rd} = \frac{2\eta W f_{yd}}{\Delta} \qquad \text{...fixed model} \qquad \text{(bis 9)}$$

$$V_{pl,Rd} = \frac{\pi d^2}{4} \cdot \frac{f_y}{\gamma_n \gamma_m \sqrt{3}} \qquad \text{(bis 11)}$$

A minimum effective gap of 60 mm must be assumed (Liber, Stockholm. 1984).
From this graph it can be seen that rock dowels of 70 mm diameter are sufficient for a horizontal load of 311 kN.

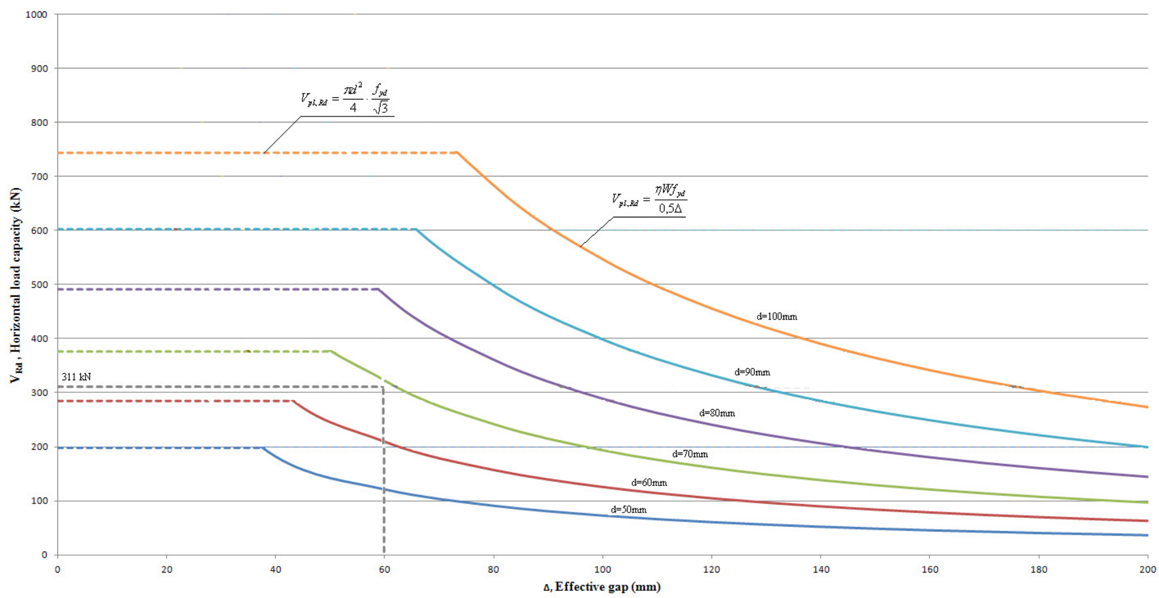


Figure 27. Horizontal load transfer capacity of rock dowels in bending according to Ryner, et al. 1996, where $f_y = 335 \times 10^3$ kN/m^2

However, according the Figure 27, the shear force capacity of a 70mm diameter bar is 376 kN. As the horizontal load is equal to the shear force acting on the section, it can be seen that shear is dominant in the section:

$$\frac{V_{Ed}}{V_{pl,Rd}} > 0,5$$

$$\frac{311}{376} > 0,5$$

Therefore a combination of shear and bending must be considered. A graph of the horizontal load capacity of bars from 50 mm to 100 mm diameter, shown in Figure 28, is calculated according to the reduced plastic bending capacity given in equation (15)
Here it can be seen that the reduced capacity of 70 mm diameter bars are insufficient. 80 mm bars are necessary to transfer the horizontal toe load of 311 kN across a minimum effective gap of 60mm.

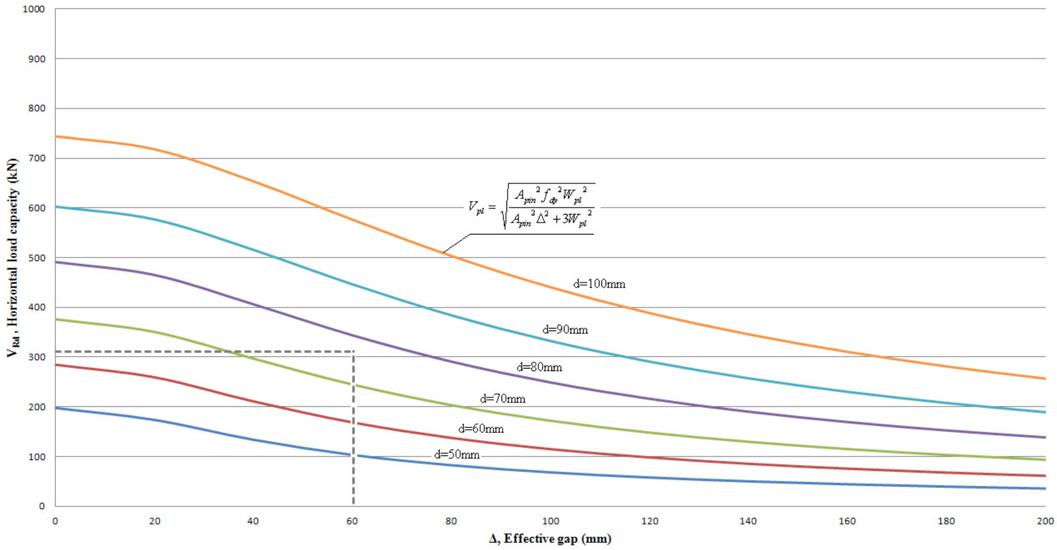


Figure 28. Reduced horizontal load transfer capacity of rock dowels in combined shear and bending, according to Kort and Karlsrud, 2008, where f_y = 335 x 10^3 kN/m^2

This design has assumed that full driving of the sheet pile to bedrock is possible. However due to the presence of the thick stony glacial till full driving of the sheet pile to bedrock may be marred. A full driving record is to be kept of the piling work and the design of the lateral toe support is to be revised and amended as the work continues. The excavation is to carried out in small section of 2,8 m width. The inclined toe bolts and toe beam are to be installed at each section as the excavation proceeds. This process will alleviate any problems due to incomplete pile driving and insufficient capacity of the rock dowels.

6.2.3 Design of drilled piles

Due to their high cost, drilled piles are only used in the deeper parts of the excavation where high earth pressures require stiffening of the sheet pile wall and high horizontal toe loads make the use of rock dowels impractical.

After the sheet pile is driven a RD400/12.5 hollow circular pile is drilled in front of the wall to a suitable embedment depth of 1m inside the bedrock. A HE240B S355 I-section is placed inside the pile and it is then infilled with C35/45 concrete as shown in Figure 29. As the excavation proceeds the drilled piles are connected to the wall on both sides of the drilled pile at 1 m intervals by welding 16mm thick S355 steel pieces with an 8 mm weld of length 200mm. The resulting section possesses a high capacity for the large horizontal loads. The resistance to bending and shear can be estimated by simply combining the section moduli and shear areas of each component as shown in Table 1.

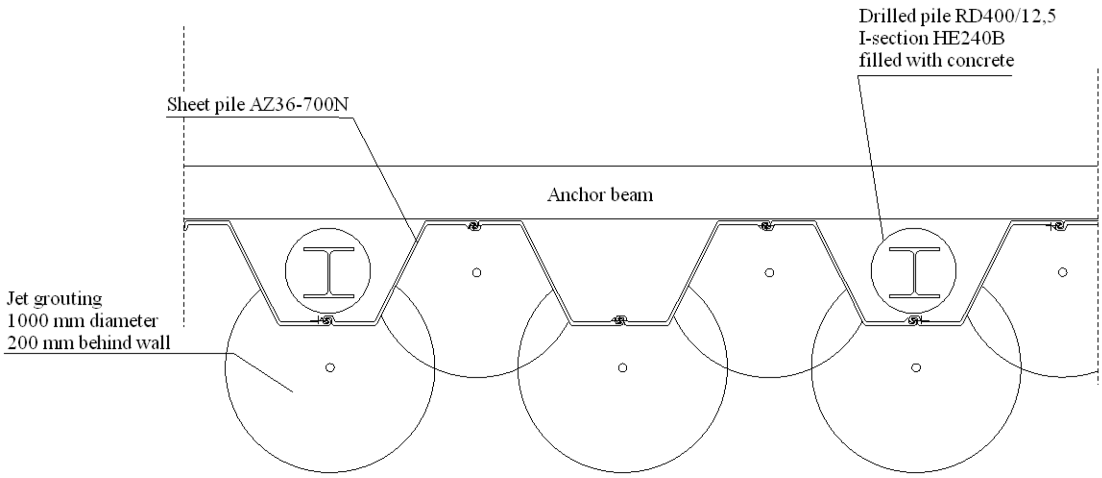
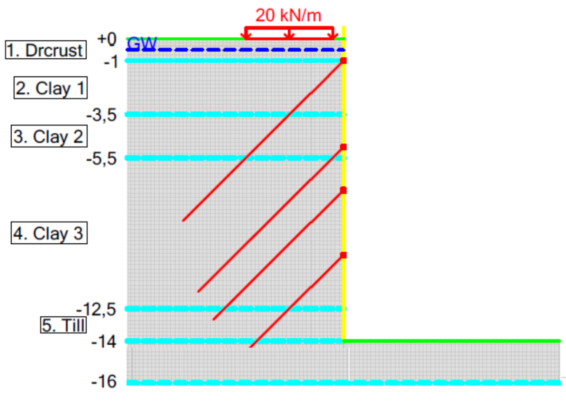


Figure 29. Plan of drilled pile supporting sheet pile wall (Finnmap Infra Oy.)

Table 1. Section properties of the drilled pile, I-Section and AZ profile

Component	W_{el}, section modulus (m^3)	A, shear area (m^2)
RD400/12.5 S355	0,0014779	0,015468
HE240B S355	0,000938	0,0106
AZ36-700N, double section	0,005030	0,03022
Total	0,0074459	0,056288

The analysis of the sheet pile wall was again carried out using Novapoint GeoCalc. GeoCalc 2.2 allows the input of user specified wall stiffness. The input data is shown in Figure 30. Due to the high stiffness of the drilled pile section, it can be assumed that negligible displacements occur at the toe of the retaining wall. A large horizontal stiffness of 2000 kN/m was specified at the toe.



Wall Type	Combined
Name	RD400/12,5 & HE240B
Manufacturer	
Cross Section Area [m^2]	0,056268
Calculation Width [m]	1,4
Inertia Modulus [m^4]	0,00166901
Section Modulus [m^3]	0,0074559
Elastic Modulus [kPa]	210000000
Flexural Stiffness [kNm^2]	350492,1
Axial Stiffness [kNm^2/m^2]	11816280

Name	A [mm^2]	L [m]	α [°]	h [m]	F [kN]	Elastic Modulus [kPa]	Overdig [m]	Horizontal distribution [m]
Level 1	2000	10,5	45	1	150	210000000	0	2,8
Level 2	2000	9,5	45	5	200	210000000	0	2,8
Level 3	2000	8,5	45	7	250	210000000	0	2,8
Level 4	2000	7,5	45	10	250	210000000	0	2,8

Id	Layer Name	z [m]	h [m]	γ [kN/m³]	Φ [°]	c [kPa]	Δc [kPa/m]	K0ud	d/ud	Material Model	δya	δyp	ξ50a	ξ50p	m	n	k
1.	Drcrust	0	1	16	0	20	0	1	Undrained	MCM					40	1	1
2.	Clay 1	-1	2,5	16	0	5	0	1	Undrained	MCM					40	1	1
3.	Clay 2	-3,5	2	16	0	7	0	1	Undrained	MCM					40	1	1
4.	Clay 3	-5,5	7	17	0	11	0	1	Undrained	MCM					40	1	1
5.	Till	-12,5	1,5	20	34	0	0	0,44	Undrained	MCM					200	0,5	0,5
6.	Rock	-14	2	20	45	10	0	0,29	Drained	MCM					10000	1	1

Figure 30. Input data for the analysis of a sheet pile wall using Novapoint Geocalc version 2.2.

The output data yielded in the analysis included the shear force distribution along the wall. The shear force distribution, representing the shear force per unit of composite section is shown in Figure 31. The maximum shear force occurring along the section is 805 kN acting at the toe.

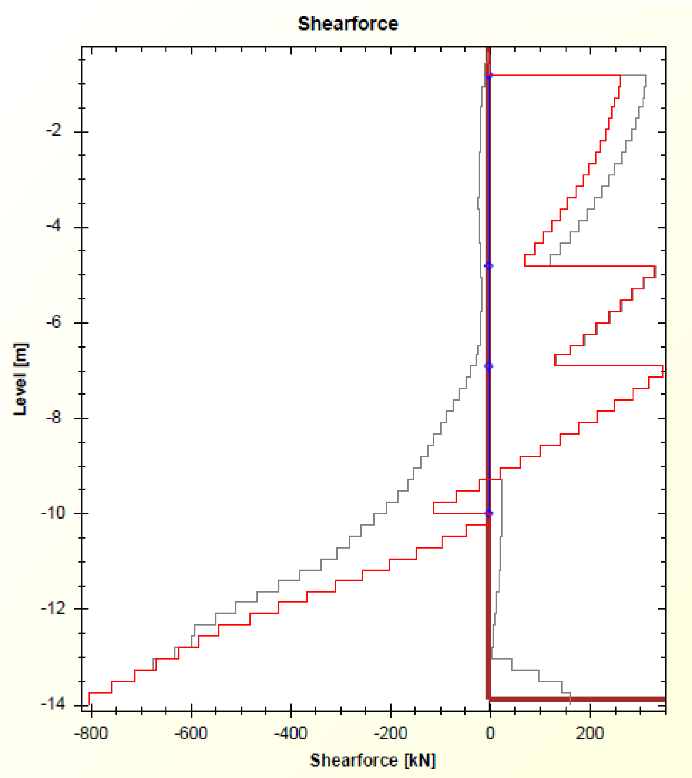


Figure 31.Output data from the analysis of a sheet pile wall using Novapoint Geocalc version 2.2.: shear force distribution along the sheet pile wall

The shear capacity of the section may be calculated according to equation (22). The yield stress of the steel is 355 x 10^3 kN/m^2.

$$V_{pl,Rd} = A \cdot \frac{f_y}{\sqrt{3}} \tag{22}$$

where V$_{pl,\,Rd}$ is the shear force capacity of the composite section [kN]

A is the total combined shear area as given in Table 1 [m^2]

$$V_{pl,Rd} = 0{,}056288 \cdot \frac{355 \cdot 10^3}{\sqrt{3}} = 11536 kN$$

An overall factor of safety of 1,5 is applicable for the temporary sheet pile wall. Then the section sufficient in shear when:

$$1{,}5V_{Ed} < V_{pl,Rd}$$

1,5 x 805 kN < 11536 kN

1207,5 kN < 11536 kN

Also include in the output data is the moment distribution along the wall. The moment distribution, representing the bending moment per unit of composite section is shown in Figure 32. The maximum bending moment occurring along the wall is 1537 kN acting just above the lowest anchor level.

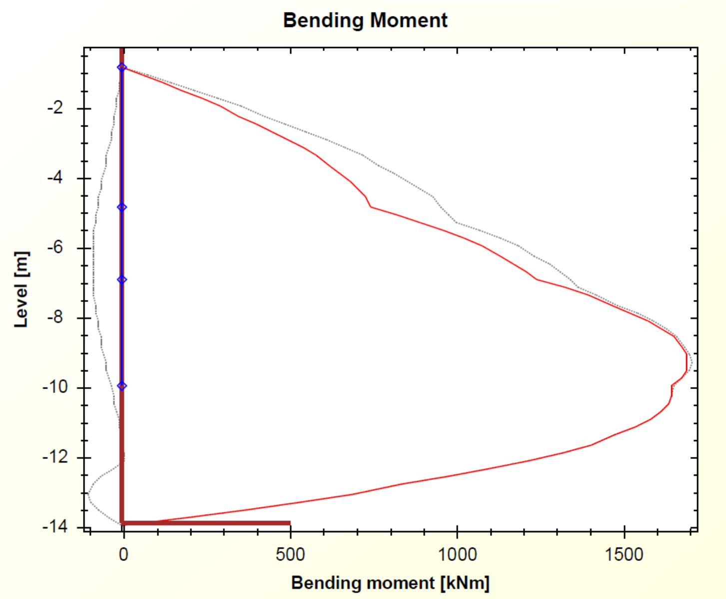


Figure 32. Output data from the analysis of a sheet pile wall using Novapoint Geocalc version 2.2.: moment distribution along the sheet pile wall

The elastic moment capacity of the section may be calculated according to equation (23).

$$M_{el,Rd} = W_{el} f_y \tag{23}$$

where $M_{el,Rd}$ is the elastic moment capacity of the combined section [kNm]

W_{el} is the total combined elastic section modulus as given in Table 1 [m^3]

$$M_{el,Rd} = 0{,}0074459 \cdot 355 \cdot 10^3 = 2643 kNm$$

The section is sufficient in bending when:

$$1{,}5 M_{Ed} < M_{el,Rd}$$

1,5 x 1537 kNm < 2643 kNm

2305,5 kNm < 2643 kNm

6.3 Sealing at the base of the excavation

The sensitivity of the nearby soil bearing structures to settlements due to groundwater lowering places strict restrictions of the disturbance allowed to the groundwater in the area surrounding the excavation. While it was found the permeability of the glacial till is quite low, a watertight barrier is required at the toe to minimize seepage beneath the toe of the sheet pile and to avoid any problems of excessive seepage through larger undetected conductive layers. Jet grouting is to be used to provide a watertight barrier as it is the most suitable grouting method for the tight glacial till layer at the toe of the wall. Other measures must also be taken. Full and complete pile driving to the bedrock and the immediate installation of the toe beam on reaching the bedrock will also minimize seepage beneath the toe. An infiltration system is to be installed in the area surrounding the excavation to supply water to the ground should the groundwater fall below critical limits.

The jet grouting is to be installed after sheet pile and drilled piles are in place. The grout is to be injected 200 mm behind wall as shown in Figure 29, in overlapping columns 1000mm in diameter and at 700 mm centers. Thus a continuous barrier to groundwater flow is formed with no pathway for groundwater seepage existing between wall and grouted soil mass.

To install the jet grouting, a borehole is to be drilled to a depth of 1m below the bedrock surface. As the drill head is withdrawn, a mortar consisting of Ordinary Portland cement is injected into the rock and the soil. The column of grouted soil is to extend 0,5m above toe, thus sealing the gap between the sheet pile toe and the bedrock.

The quality of the jet grouting work is to be tested prior to excavation. Cores are extracted from the grouted mass and tested for strength of 6 MPa at 28 day to confirm the development of the grout in the soil. In addition pumping tests and continued monitoring to the groundwater level by the system of groundwater monitoring tubes surrounding the excavation indicated of the performance of the groundwater exclusion system.

7 Conclusions

The use of rock dowels to provide lateral support at the toe of a retaining wall extending to bedrock is widely described in literature and commonly used in practice. Problems have been encountered regarding the use of rock dowels in Finland. In these cases rock dowels have not provided a secure restraint to the toe of the wall due to the incorrect positioning of the rock dowels. This problem is associated with the use of bored installation tubes for the installation of the dowels. The large errors involved in boring the installation tubes from the ground surface to bedrock occasionally result in a large space between the wall and the rock dowel allowing excessive displacement at the toe of the wall. Installation of the rock dowels through casings attached to the wall eliminates this problem. For this reason the use of casings attached to the wall is the most recommended method of rock dowel installation in literature and is the only method described in Eurocode SFS EN 12063:1999.

However casings attached to the wall are not always favored in practice due to the difficulties in ensuring the casing is not damaged during pile driving. Additionally the extra handling involved in attaching the casings entails extra cost, making this method more expensive.

It is also considered that the installation of rock dowels through a bored installation tube does not cause any problems if the excavation is executed carefully in small sections and ensuring the stability of each section before proceeding to the next. In this way any rock dowels found to be insufficient can be replaced or the inclined toe bolts and concrete toe beam can be installed to ensure sound toe support.

In Finland the design procedure for rock dowels given in Pohjarakennus RIL 95 is commonly used. This simple design procedure considers only shear forces acting on the dowel. A more detailed design taking into account bending of the rock dowel due to a gap between the toe of the sheet pile and the rock has not been considered necessary. The large factor of safety applied in the design given in Pohjarakennus RIL 95 is assumed to take account gaps of up to 200 mm. A careful excavation process, proceeding in small section and ensuring the stability of each section can be used to compensate for the under-design of any dowels encountered.

The requirements for sealing at the toe of a retaining wall extending to bedrock depend on the site conditions. A concrete toe beam is the most widely used method of seepage control at the toe of the wall. It is suitable in conditions of low groundwater seepage beneath the toe of the wall on excavation to bedrock where the occurrence of some seepage beneath the toe is allowable.

Casting of a concrete toe beam becomes difficult in conditions of high groundwater seepage beneath the toe of the wall. In this case grouting techniques must be employed as a groundwater control measure. Penetration grouting has been used in Finland with varying success. However it is generally not suitable for the soil conditions overlying the Finnish bedrock. Therefore penetration grouting is a relatively unreliable method for sealing the toe of a retaining wall extending to bedrock. Jet grouting is to be considered the most dependable technique, suitable for all soil types. Therefore jet grouting is highly recommended where sealing at the toe is crucial.

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List of Appendices

Appendix 1. List of Interviewees

Gustavson, Harry. 2011, Arcus Ltd., Civil Engineering

Korhonen, Osmo 2011, Helsingin kaupungin geotekninen osasto

Kärki, Tuomas 2011, FCG

Länsivaara, Tim 2011 , TTY

Pakarinen, Mika 2011, YIT

Rahikainen, Teemu 2011, FCG

Ranta-Aho, Tapio. 2011, Sito

Vehmas, Hari 2011, FCG

Viitala, Jouko. 2011, Lemmikäinen

Virkkunen, Pentti 2011

Vunneli, Juha. 2011, YIT

Väätäinen, Risto. 2011, YIT